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# Hydrograph Synthesis for Watershed Subzones from Measured Urban Parameters

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HYDROGRAPH SYNTHESIS FOR WATERSHED SUBZONES  
FROM MEASURED URBAN PARAMETERS

by

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## ABSTRACT

### HYDROGRAPH SYNTHESIS FOR WATERSHED SUBZONES FROM MEASURED URBAN PARAMETERS

An analog computer program was developed to simulate the outflow hydrographs at four locations within the 38th Street Waller Creek urban watershed at Austin, Texas. Actual outflow was gaged at the final outlet of the watershed. This provided a checkpoint for comparing the simulated and observed final outflow hydrographs.

The outflow hydrographs for each subzone were obtained by chronologically abstracting interception, infiltration, and depression storage from their precipitation hyetographs. These outflow hydrographs were then routed through Waller Creek channel to obtain the hydrographs at the four desired locations.

The advantages of this model are the flexibility in varying the precipitation inputs to each subzone and the ability to obtain the contribution to the final flood hydrograph of each subzone.

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KEYWORDS-\*urban hydrology/ simulation/ \*simulation of urban hydrology/ hydrologic models/ watershed studies/ hydrology/ hydrologic research/ computer simulation/ \*electronic analog computer/ surface runoff/ storm drain design/ flood frequency/ \*urban parameters/ equivalent rural watershed/ \*runoff characteristics.

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## PARTIAL LIST OF SYMBOLS

$b$	channel width	$Q_i$	rate of inflow into the upstream section of a channel
$C_f$	percentage impervious cover	$Q_o$	rate of outflow from the downstream section of a channel
$d$	length along a section of channel	$Q_P$	peak discharge
$f$	instantaneous capacity rate of infiltration	$Q_T$	total volume of outflow
$f_c$	average minimum infiltration rate over the entire catchment area	$S$	catchment area storage
$f_o$	average maximum infiltration rate over the entire catchment area	$S_d$	total volume of available depression storage expressed as a mean depth over the entire catchment area
$i$	rate of precipitation	$S_I$	volume of interception storage capacity expressed as an average depth over the entire catchment area
$i_{cc}$	capacity rate of inflow into interception storage	$SW_i$	the $i^{th}$ subwatershed
$L_f$	characteristic impervious length factor	$t$	time
$L_m$	mean characteristic impervious length	$t_D$	storm duration
$n$	Manning's roughness coefficient	$T_L$	lag time of water flowing between upstream and downstream channel sections of a subzone
$P$	cumulative precipitation	$t_R$	rise time
$P_e$	rate of rainfall excess	$\sigma_c$	capacity rate of inflow into depression storage
$P_i$	refers to the $i^{th}$ potentiometer		
$Q$	rate of runoff		

watershed. Because the study areas were all 100 percent impervious, infiltration and interception were neglected, and depression storage accounted for all abstractive losses.

$$\dot{q} = \frac{ds}{dt} \quad (1.1)$$

i = inflow rate (effective precipitation)  
q = outflow rate (rate of runoff)

$$q = q_{\max} \exp(-\tau/k) \quad (1.2)$$

$$q_{\max} = i(1 - e^{-1/k}) \quad (1.3)$$

$k$  = parameter indicating lag time

$\tau = t - 1$

Schaake, Jr. (1965), also studied the drainage characteristics of small impervious areas. These small impervious areas produce runoff to storm water inlets. A small impervious area is divided into areas of overland flow which drain into areas producing swale flow. Ultimately the swale flows drains into a storm inlet.

Willeke (1966) hypothesized that a linear storage system could be used to transfer the time distribution of effective precipitation into the time distribution of runoff. If this idealization is sufficiently accurate, then the lag time of the watershed would be the only parameter needed to describe the characteristics of the linear storage system.

Evidence presented showed that the lag time, defined as the time between centers of mass of effective precipitation and runoff, is essentially constant for small urban watersheds. It was also shown that on the watersheds studied, there were few significant correlations between lag time and storm magnitude. These findings tend to support the hypothesis of linearity.

Espey, Jr., Morgan, and Masch (1965) undertook a study to evaluate the various effects of urbanization on the hydrologic characteristics of a small urban watershed located within Austin, Texas. A linear regression analysis of data from 24 urban and 11 rural watersheds was used to derive equations which would evaluate the past rural conditions and predict future urban conditions for the Waller Creek watershed. The effects of urbanization on the discharge hydrograph include a decrease in hydrograph rise time, as well as increases in peak discharge and unit yield ( $\text{in}/\text{mi}^2$ ).

For determining the 30-minute unit hydrograph for rural watersheds, the following equations were developed from data of 11 rural watersheds in Texas, Oklahoma, and New Mexico.

$$T_{RR} = 2.65 L^{0.12} S^{-0.52} \quad (1.4)$$

The expressions for the 30-minute hydrograph for urban watersheds, based on data from 22 watersheds throughout the United States follow:

approaches have been taken in this field; however, several major techniques predominate. The use of the unit hydrograph method after chronologically abstracting losses is widely accepted. Studies on a micro-scale, where overland flow equations are applied to small impervious areas, are also used. Finally, statistical analysis of runoff and pertinent parameters provide another major avenue of investigation.

Tholin and Keifer (1960) made a detailed study of rainfall-runoff relationships in urban areas based on a "Design Storm" of three hours duration. The "Chicago Hydrograph Method" of sewer design evaluates, in detail, the physical affect of rainfall abstractions and flow detentions which intervene between the hyetograph of rainfall and the hydrographs of sewer supply and sewer outflow. Various types of uniform land use, ranging from suburban residential to industrial and commercial, were analyzed to determine their influence on the infiltration capacity of pervious areas; depression storage; overland flow detention; detention in gutters; and detention in lateral sewer systems.

In evaluating the land use characteristics in terms of pervious and impervious areas several simplifying assumptions were introduced.

1. Public and private sidewalks draining onto pervious areas were considered as part of the pervious area. This was justified by the increase in infiltration capacity caused by the lateral spread of water under the walks.
2. Building roofs, which are directly connected to the sewer, are converted into a uniform strip of equivalent area. This allows one to assume an average length of overland flow from pervious areas in a uniform series of blocks.
3. The garage roofs in residential areas, because they discharge onto backyard lawns, were considered a part of the pervious areas.

Utilization of the Stanford Watershed Model in studies where a complete water budget is of interest has produced some significant results. This digital computer model is based on a system of equations which keep a running tabulation of all water entering the watershed as precipitation, stored within the watershed and leaving the watershed as runoff, subsurface outflow, or evapotranspiration (Crawford and Linsley, 1962).

James (1965) used the Stanford Watershed Model to develop a long-term continuous hydrograph between 1905 and 1963 for Morrison Creek, Sacramento County, California. By varying constants which describe the physical conditions within the watershed according to the amount of urban development and channel improvement within the tributary

area, a number of continuous hydrographs were developed. Synthesis of hydrographs for any combination of urbanization, channel improvement, and precipitation input could, therefore, be produced. Morrison Creek drains 72.7 square miles upstream from the Western Pacific Railroad just south of Sacramento.

James attempted to relate the independent variables (urbanization, channelization, and tributary area) with the dependent observed flood peaks. A trial and error procedure was used to determine the 32 constants and 3 arrays necessary to describe the watershed characteristics for the digital program. In his models, the following inputs were altered to represent changes in the degree of urbanization:

1. Advance of the time-area histogram of inflows to reflect the probable installation of storm sewers;
2. Increase in the impervious area;
3. Decrease in maximum hourly interception and depression storages because of the reduction in undrained natural depressions with increase in urbanization;
4. Reduction in upper zone soil moisture storage;
5. Reduction in overland flow delay; and
6. Reduction in interflow delay.

James concluded from his study that impervious cover and channelization are the most important parameters of urbanization significantly affecting the runoff process in a watershed.

Viessman, Jr. (1966), considered the design of a storm drainage system as a twofold problem: (1) individual runoff inputs must be determined at each inlet or group of inlets; and (2) these inputs must then be routed and combined in the storm sewers, enabling outflow hydrographs to be synthesized at any point of interest. His study is limited, specifically, to the determination of runoff hydrographs for small (0.4 to 1.0 acre) impervious urban inlet areas. In small urban areas (up to several acres in size) the drain can be considered to act as a linear reservoir. The hydrology of these small areas is exceedingly important, as most composite urban drainage areas can be broken up into a subset of smaller areas usually tributary to the storm water inlets. Thus, if the runoff hydrograph at each inlet within an urban drainage area can be predicted adequately, then routing of the various inlet flows would produce the desired outflow hydrograph.

In his study, Viessman, Jr., used the following method of analysis:

1. The rainfall excess was determined by considering the same losses present in a rural

$$W_{75U} = 1.34 \times 10^4 A^{0.92} Q^{-0.94} \quad (1.13)$$

in which

$T_R$  = rise time of the unit hydrograph  
 $I_R$  = percentage impervious cover  
 $Q$  = peak discharge  
 $T_B$  = time base  
 $\Phi_B$  = urbanization factor reflecting degree of channelization  
 $W_{50}$  and  $W_{75}$  = hydrograph widths at 50 and 75 percent of the peak discharge respectively  
 $U$  = urban watersheds  
 $R$  = rural watersheds

The writers felt that additional urbanization data were needed to develop more reliable and general relationships applicable to both urban and rural watersheds.

Several conclusions based on the urban hydrology literature reviewed are apparent.

1. More research investigating the effects of intermingling pervious and impervious areas is needed.

2. The unit hydrograph method is at present the most versatile approach to hydrograph synthesis of excess rainfall.

3. Lack of sufficient data is holding back more conclusive results in statistical studies.

4. Accurately describing the abstractive processes of interception, infiltration, and depression storage is the most difficult aspect of urban watershed modeling.

5. The "rational method," with its use of an all encompassing runoff coefficient, oversimplifies the complexity of urban drainage design.

In that the analog computer simulation of an urban watershed made by Narayana (1969) forms the basis of the present study, it will be discussed in the next chapter.



## Chapter II

### DEVELOPMENT OF THE PHYSICAL MODEL

The research undertaken in this study makes direct use of the urban watershed model developed by V. V. Dhruva Narayana (1969) in two different circumstances. In the first case, Narayana's model is applied, without modification, to the Wilbarger Creek rural watershed. The parameters used to quantify the runoff characteristics of the rural watershed are adjusted to reflect the effect of no impervious cover.

The second case involves applying the urban watershed model to small sections (subzones) of the original watershed, and by routing outflows of these subzones, reproduce the final outflow hydrograph. The ability to input arbitrary rainfall hyetographs to each subzone and obtain the effects at several different locations within the watershed provides a useful improvement of the original urban watershed model.

#### Modeling Procedure

The study made by Narayana (1969) sought to accurately describe the runoff process of an urban watershed by developing an adequate mathematical model which could be programmed on an analog computer for verification. The watersheds used in the study were the 23rd and 38th Street Waller Creek Watersheds in Austin, Texas. These watersheds underwent progressive urbanization during the period of observed runoff data.

Narayana's (1969) modeling procedure is outlined below.

1. Identification and definition of measurable urban parameters.
2. Mathematical description of the various phases of the runoff process in terms of the physical characteristics of the watershed (watershed coefficients).
3. Verification of the mathematical model on an analog computer by simulation of several recorded runoff events.
4. Determination of the watershed coefficients from model verification and relating them to the corresponding urban parameters.
5. Prediction of future urban parameters and determination of the corresponding watershed coefficients.

Use of the verified model would then require knowledge of only two parameters.

1. Estimates or exact values of the urban parameters in the year for which the streamflow is to be predicted.
2. Estimated or assumed design storm hyetographs.

Utilizing the functional relationship developed in step 4 between watershed coefficients and urban parameters the values of watershed coefficients could be determined from any given set of urban parameters. Using the watershed coefficients just determined, the outflow hydrograph could be predicted for any appropriate storm hyetograph.

#### Urban Parameters

The characteristics of urbanization considered in the study were (1) the percentage impervious cover, and (2) the characteristic impervious length factor.

The percentage impervious cover,  $C_f$ , is defined as the ratio of the total impervious area (area covered by roofs, roads, parking lots, sidewalks, etc.) to the total catchment area. This factor is an index which characterizes the changes in abstractive processes that ultimately alter the time distribution and total volume of rainfall excess.

The characteristic impervious length for a particular impervious element (area  $a_i$ ) of a catchment area is defined as the length of travel,  $l_i$ , between the center of the area and the discharge measuring point (Figure 2.1).

The mean characteristic impervious length,  $L_m$ , for the catchment area is given by

$$L_m = \frac{\sum a_i l_i}{\sum a_i} \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad (2.1)$$

in which

- $l_i$  = length of travel of water draining from the center of the  $i^{\text{th}}$  impervious element, to a specified measuring point
- $a_i$  = area of the  $i^{\text{th}}$  impervious element

The characteristic impervious length factor,  $L_f$ , for the catchment area is given by

$$\text{in which } L_f = \frac{L_m}{L} \quad . \quad . \quad . \quad . \quad . \quad (2.2)$$

$L$  = the maximum length of travel of water draining from the catchment area (Figure 2.1)

The parameter,  $L_f$ , is, therefore, an index of the location of the impervious areas with respect to the final outflow point of the catchment area. Clearly, the travel time of the rainfall excess from its point of origin to the final outflow point is a function of this factor.

#### Determination of Rainfall Excess

##### Precipitation

The initial step in watershed modeling is to determine representative storm hyetographs for the catchment area under investigation. Applying the isohyetal or Thiessen's techniques to a rain gage network, one obtains the aerial distribution of precipitation.

##### Interception

The rate of interception was assumed to reduce exponentially with an increase in moisture captured in interception storage.

$$i_{cc} = i e^{-P/S_I} \quad . \quad . \quad . \quad . \quad (2.3)$$

in which

$i_{cc}$  = capacity rate of inflow into interception storage  
 $i$  = rate of precipitation  
 $P$  = cumulative precipitation  
 $S_I$  = volume of interception storage capacity expressed as an average depth over the catchment area

The net precipitation after satisfying interception can be expressed by the following equation:

$$i_1 = i \left( 1 - e^{-P/S_I} \right) \quad . \quad . \quad (2.4)$$

The major portion of moisture accumulated in interception storage is lost through evaporation. However, this evaporation loss does not form a significant mechanism of the runoff process for short duration storms.

##### Infiltration

Narayana (1969) chose Horton's equation which expresses infiltration capacity as a function of time.

$$f = f_c + (f_o - f_c) e^{-k_f t} \quad . \quad . \quad (2.5)$$

in which

$f$  = the instantaneous capacity rate of infiltration  
 $t$  = the time measured from the beginning of the infiltration curve  
 $f_c$  = the constant rate at which  $f$  is approached asymptotically with time  
 $f_o$  = the initial rate at  $t = 0$   
 $k_f$  = a positive coefficient depending upon the soil characteristics

As indicated by Figure 2.2, the actual infiltration rate,  $f_a$ , curve follows the hydrograph of net precipitation,  $i_1$ , until the net rainfall intensity (rainfall less interception) exceeds the infiltration rate capacity curve.

The initial infiltration capacity rate is located at a value,  $f_{t1}$ , less than the maximum of the capacity rate curve, by sliding the curve to the left. The actual value  $f_{t1}$  depends upon the prevailing soil moisture status.

##### Surface depression storage

The capacity rate of inflow into depression storage is expressed by the following equation:

$$\sigma_c = i_2 e^{-(P_1 - F)/S_d} \quad . \quad . \quad (2.6)$$

in which

$i_2 = i_1 - f$  = net rate of precipitation after satisfying interception and infiltration  
 $P_1$  = accumulated rainfall having satisfied interception storage  
 $F$  = accumulated infiltration loss  
 $S_d$  = total volume of available depression storage (expressed as mean depth over the entire catchment area)  
 $\sigma_c$  = capacity rate of inflow into depression storage

##### Hydrograph of rainfall excess

The hydrograph of rainfall excess is computed by chronologically deducting the losses due to interception, infiltration, and depression

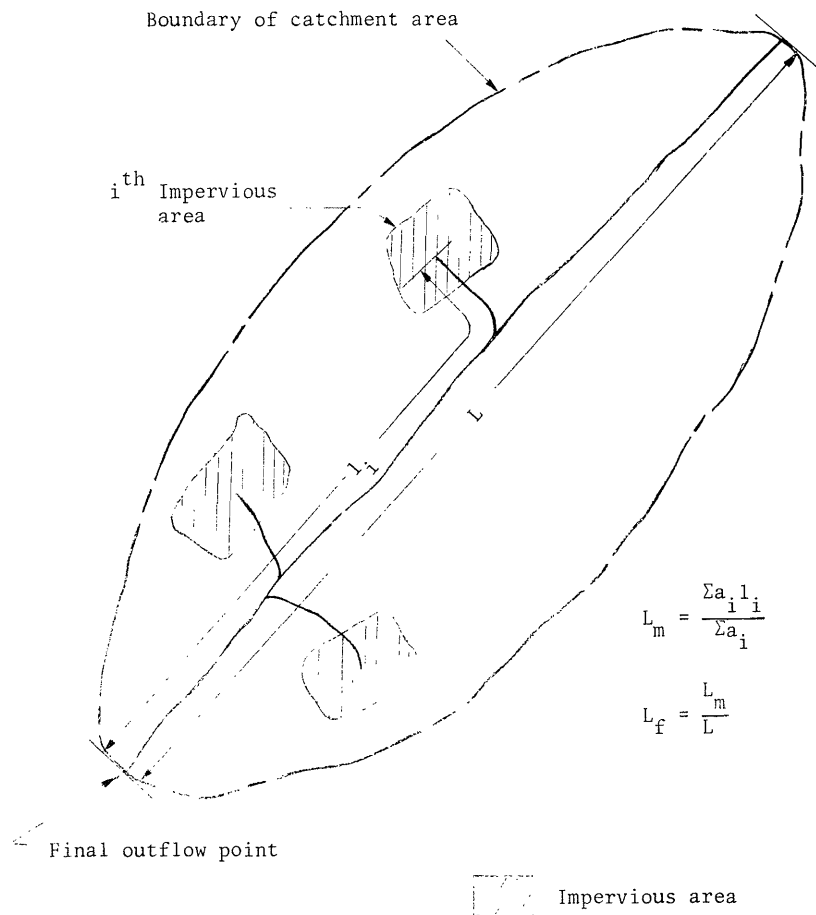


Figure 2.1. Sketch illustrating the characteristic impervious length,  $l_i$ .

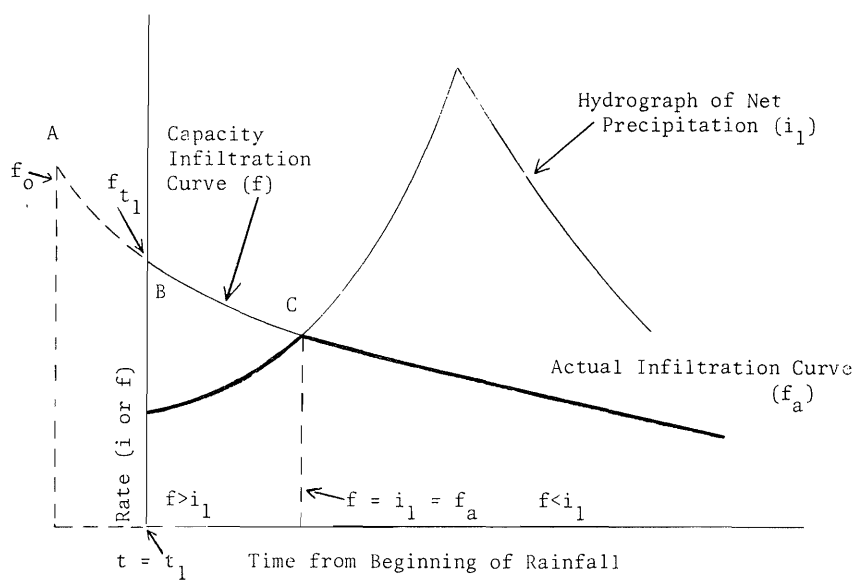


Figure 2.2. Typical actual infiltration rate curve.



storage from the hydrograph of precipitation in compatible and finite time increments. The schematic flow chart of this procedure along with the various equations developed so far are presented in Figure 2.3.

The general continuity equation for any linear storage system is given below.

$$P_e - Q = \frac{ds}{dt} \quad (2.7)$$

in which

- $P_e$  = rainfall excess rate
- $Q_e$  = runoff rate
- $S$  = catchment area storage (overland and channel components)

Catchment area storage was assumed as directly proportional to the outflow rate.

$$S = t_R Q \quad (2.8)$$

### Overland-Flow Routing

Narayana (1969) adopted the simple procedure of "storage routing" wherein the storage effects (overland and channel components) of the catchment area are accounted for by the characteristic time lag of the catchment area.

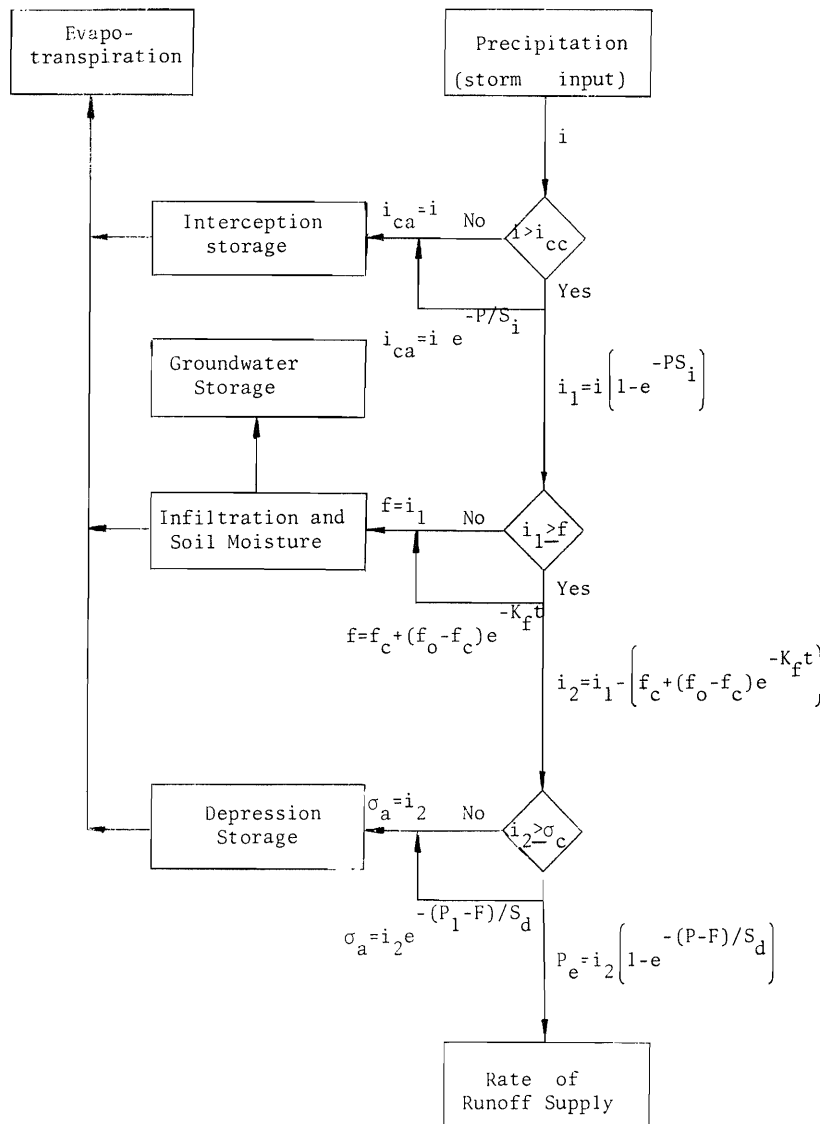


Figure 2.3. Schematic flow chart for obtaining hydrograph of rainfall excess.

in which

$t_R$  = the proportionality factor approximated by the hydrograph rise time

Using the equations derived by Espey, Jr., Morgan, and Masch (1965), the rise time is expressed as a function of the length of travel,  $L$ , and the mean slope,  $S$ , of the catchment area. Hence,

$$P_e - Q = t_R \frac{dQ}{dt} \quad . \quad . \quad . \quad (2.9)$$

$Q$  can be obtained by solving this differential equation.

#### Subwatershed Model

A clear concept of the urban watershed model as applied to a subzone or subwatershed is essential at this point. The model's original use was confined to simulating the runoff from an entire watershed at its final outflow point. This study divides the watershed in subwatersheds to which the model is applied individually. The outflows of each subwatershed are then routed together to produce the outflow at selected points (A, B, C, D, Figure 2.4) downstream.

D, Figure 2.4) downstream.

To reiterate, in each catchment area to which the model is applied, there exists no ungaged inflow and all outflow is assumed to exit at one point of the channel flowing from the area (Figure 2.4). In reality it may require judicious selection of the subzone boundaries in order to satisfy the assumption of all outflow exiting through one point of each subzone.

#### Channel Routing

Applying the urban watershed model on a subzone level produces the outflow hydrographs at the discharge points of each subzone. Obviously some technique of channel routing must be devised to combine each of the subzone discharges to produce the final hydrograph of the entire basin.

The same linear reservoir storage method used for overland-flow routing is assumed to be valid. Hence,

$$Q_i - Q_o = \frac{dS}{dt} \quad . \quad . \quad . \quad (2.10)$$

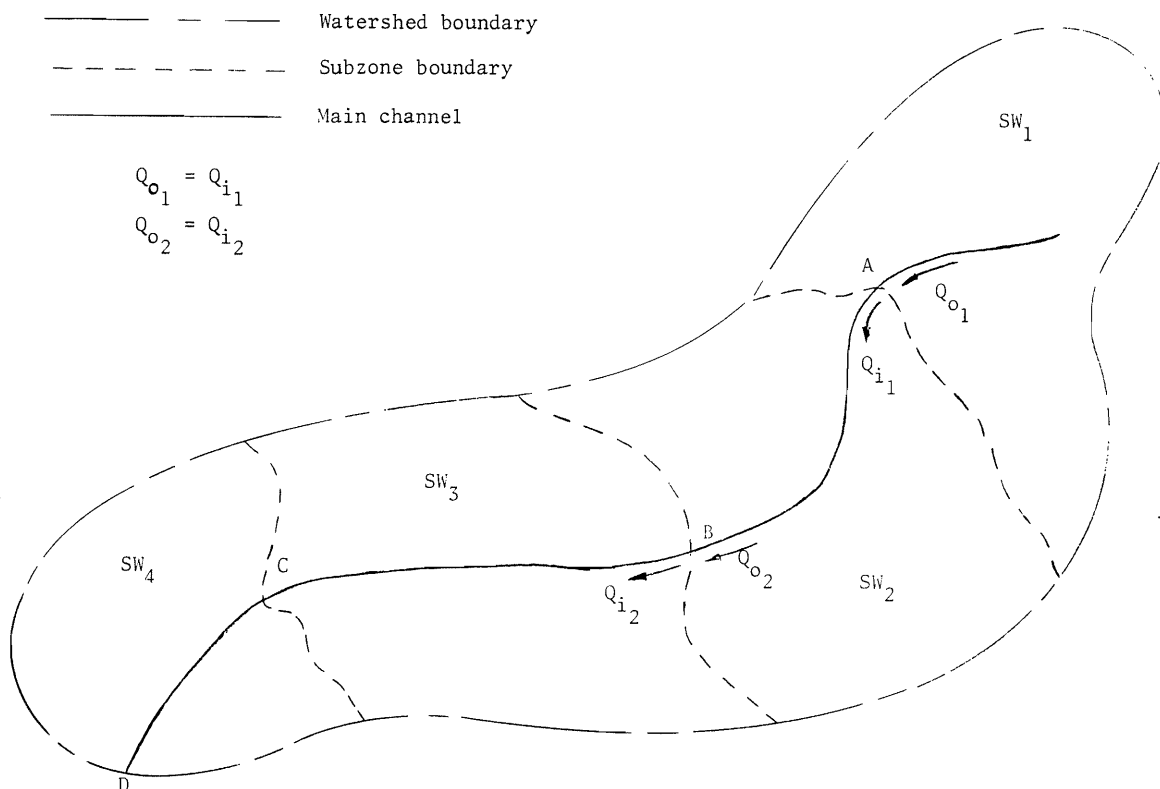


Figure 2.4. Schematic diagram of the subwatershed model.

and

$$S = T_L Q_o \quad . \quad . \quad . \quad . \quad . \quad (2.11)$$

in which

- $Q_i$  = rate of inflow into the upstream section of channel (Figure 2.4), in this instance the upstream section coincides with the upstream boundary between subzones
- $Q_o$  = rate of outflow from the downstream section of channel which coincides with the boundary between the two adjacent downstream subzones
- $S$  = instantaneous volume of channel storage
- $T_L$  = proportionality factor between  $S$  and  $Q_o$  which represents the time lag of water flowing between upstream and downstream channel sections

The resulting equation is given below.

$$Q_i - Q_o = T_L \frac{dQ_o}{dt} \quad . \quad . \quad . \quad (2.12)$$

An expression for time lag,  $T_L$ , will be derived in the next section from consideration of Manning's equation.

#### Derivation of Lag Time

The use of a linear storage system analogy for channel routing in the subwatershed model necessitates the derivation of an expression for the characteristic lag time,  $T_L$ , in the following equation.

$$Q_i - Q_o = T_L \frac{dQ_o}{dt}$$

A rectangular channel cross section is assumed throughout the watershed in order to simplify analysis.

$b$  = channel width in feet

and

$y$  = depth of flow in feet

therefore

$A = by$  = cross sectional area of flow

and

$P = b + 2y \approx b$  = the wetted perimeter of flow

Manning's open channel flow equation is given below (Henderson, 1966).

$$Q = VA = 1.49 \frac{A R^{2/3} S^{1/2}}{n} \quad . \quad . \quad . \quad (2.13)$$

in which

- $Q$  = discharge in cfs
- $S$  = channel slope in ft/ft
- $n$  = Manning's roughness coefficient
- $R$  = hydraulic radius
- $R^{2/3} = (A^{2/3}) / (P^{2/3}) \approx y^{2/3}$

Therefore,

$$Q = 1.49 \frac{(by) y^{2/3} S^{1/2}}{n} = 1.49 \frac{b S^{1/2}}{n} y^{5/3}$$

Solving for  $y$  as a function of  $Q$

$$y = f(Q) = \left( \frac{n}{1.49 b S^{1/2}} \right)^{3/5} Q^{3/5} \quad . \quad (2.14)$$

$$y = K Q^{3/5} \quad . \quad . \quad . \quad . \quad . \quad (2.15)$$

in which

$$K = \left( \frac{n}{1.49 b S^{1/2}} \right)^{3/5} \quad . \quad . \quad . \quad . \quad (2.16)$$

The following derivation leads to an expression for  $T_L$  as a function of instantaneous discharge, a quantity readily obtained from the analog circuits.

$$T_L = \frac{\text{distance}}{\text{velocity}} = \frac{lA}{Q} = \frac{lby}{Q} \quad . \quad . \quad . \quad (2.17)$$

Substituting Equation (2.16)

$$T_L = \frac{lbKQ^{3/5}}{Q}$$

$$T_L = lbKQ^{-2/5} \quad . \quad . \quad . \quad . \quad (2.18)$$

Assuming a linear distribution of inflow into the channel along its length, then a reasonable expression for  $Q$  within a subzone is given by the following:

$$Q = \frac{Q_i + Q_o}{2} \quad . \quad . \quad . \quad . \quad (2.19)$$

An expression for lag time,  $T_L$ , (Equation 2.18) in terms of readily measurable parameters has been derived.

## Chapter III

### DESCRIPTION OF THE WALLER CREEK EXPERIMENTAL WATERSHED

Waller Creek watershed in Austin, Texas, was chosen for this analog watershed modeling study for several reasons. First, the work completed on Waller Creek watershed by Narayana (1969) and Espey, Jr., Morgan, and Masch (1965) provide a solid base for more depth studies of the basin. In addition, the USDA Agricultural Research Service has aerial photos for Waller Creek from flights made in the years 1951, 1958, and 1964. The following description of Waller Creek watershed is a summary of the detailed account given by Espey, Jr., Morgan, and Masch (1965).

Waller Creek, a tributary of the Colorado River, lies entirely within the city limits of Austin, Texas (Figure 3.1). Located in the northern part of the city, the drainage area above the 23rd Street gaging station is 4.13 square miles. Above the 38th Street gage is a less urbanized subwatershed of 2.31 square miles. It is the 38th Street watershed to which the subzone model is applied (Figure 3.2).

#### Climate

The climate is generally mild and semi-humid. The ground rarely freezes, and then only at the surface. The mean annual precipitation of 33.36 inches is generally well distributed throughout the year; however, individual rainfalls of excessive amounts occur at irregular intervals.

#### Geology

Waller Creek watershed is located in the West Gulf Coastal Plain and is underlain by two bedrock formations and a thin alluvial formation. Eagle Ford Shale underlies the extreme northwestern part of the watershed while the majority of the remaining area is underlain by Austin chalk. The Austin chalk weathers to a very heavy black clay soil, which has a very low permeability. The bedrock formations are covered in the southern part by an alluvial terrace consisting of sandy highly permeable material of the ancient Colorado River.

#### Topography

The area consists of gently rolling, hilly land and is characterized by glaring white out-

crops of limestone on the slopes and in the bluffs of the creek. The maximum width of this long and narrow area is 2.6 miles at 45th Street and the minimum width is 0.9 miles near the gaging station. The average slope,  $s$ , of the main channel is 0.009 ft/ft and is fairly constant.

#### Instrumentation

The watershed's basic instrumentation consists of two streamflow stations and six rain gages (3 non-recording and 3 recording) (Figure 3.2). The two streamflow stations are equipped with standard A-10 Stevens recorders.

#### Drainage Conditions

The headwaters of Waller Creek are located south of Anderson Lane, in the northern part of the city. The main channel has been extended by excavation to the natural divide just north of Croslin Street (Figure 3.2). A drainage ditch joins the main channel just south of where it crosses Airport Boulevard. The drainage ditch was formed by the Texas and New Orleans Railroad track and this was reported to contribute additional runoff from an area of 0.3 square miles.

A second branch, called West Branch, originating in the general area of west 45th Street and Lamar, joins the main channel just west of San Jacinto Boulevard approximately two blocks above the 23rd Street stream gaging station. Beginning in the Hemphill Park area this second branch is a rock-lined channel varying in cross section from trapezoidal to rectangular in shape between 32nd Street and just south of west 30th Street where the rock lining ends.

Many small diversions are present within the natural basin caused by storm sewers and embankments.

#### Urban Parameters

It has already been established that the most difficult part of hydrograph synthesis in urban areas with mixed pervious and impervious cover, is the accurate determination of the abstractive processes of interception, infiltration, and

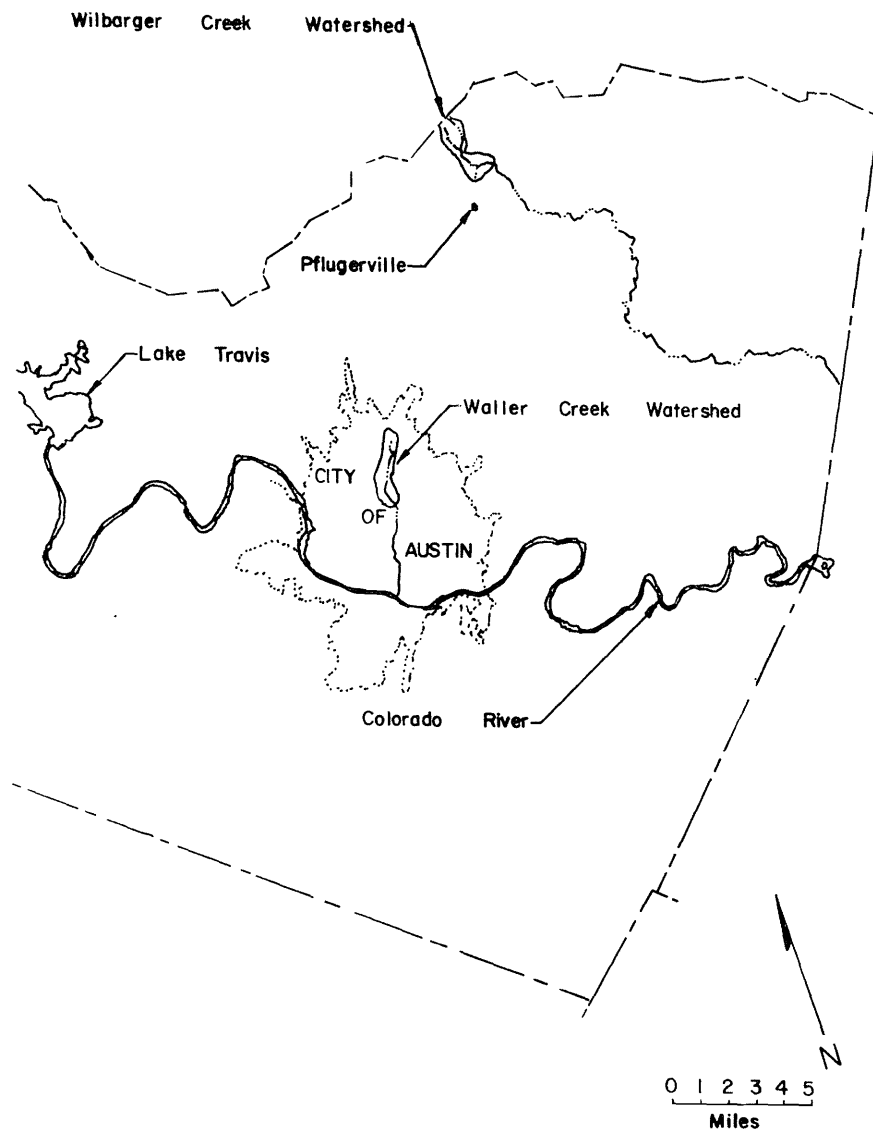


Figure 3.1. Map of Travis County showing the location of Waller Creek.

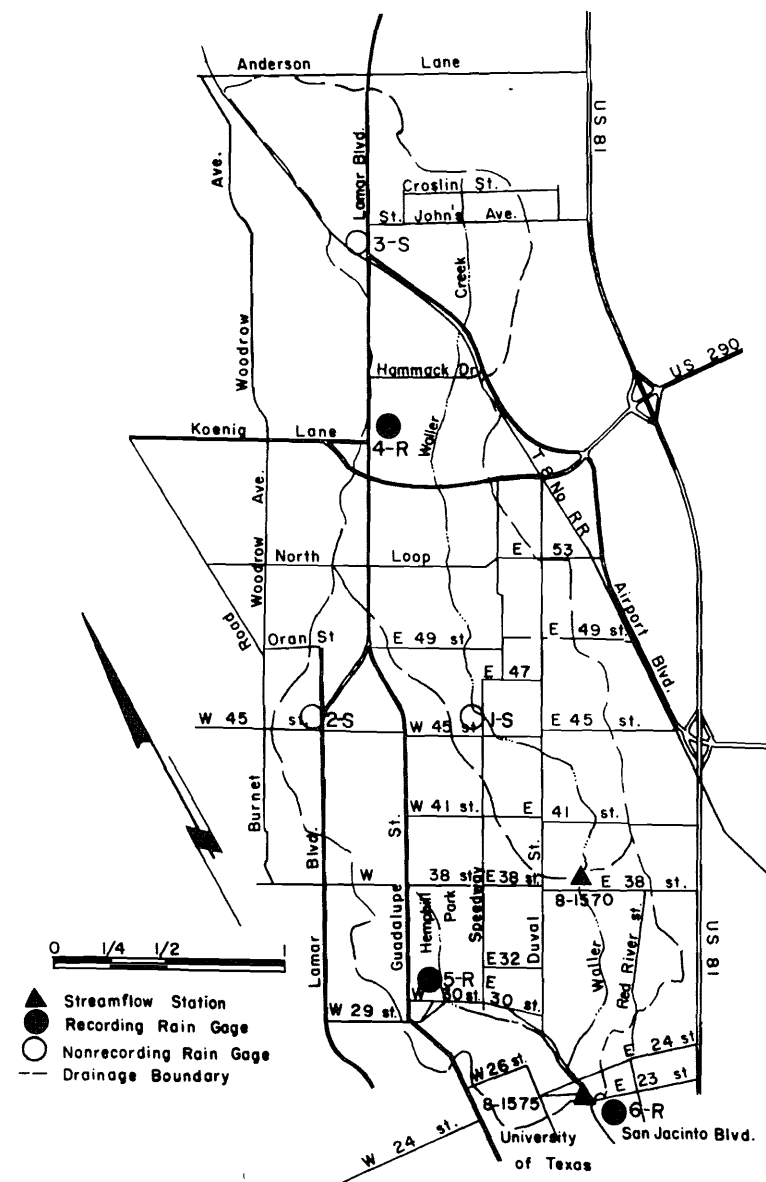


Figure 3.2. Map of Waller Creek study area showing locations of hydrologic instrument installations.

Figures 3.6 and 3.7 are graphs indicating the variation of total percentage impervious area and characteristic impervious length factor, respectively, for each subwatershed as a function of time.

Having determined the urban parameters, the watershed coefficients may be obtained for each subzone from a set of regression equations developed by Narayana (1969). Two problems still remain, however. First, Narayana's values of  $C_f$  and  $L_f$  in Table 3.2 were computed using the entire watershed as the primary spatial unit. The values of  $C_f$  and  $L_f$  in this study had to be consolidated from the values within each subzone.

Secondly, Narayana's regression equations are based on his calculations of the urban parameters. Referring to Table 3.2, one notices the disparity between the values of  $C_f$ , and the close agreement of values of  $L_f$ .

The remedy to this second problem is illustrated in Figure 3.8. Both sets of values of  $C_f$  are plotted on different axes of the graph. In order to determine the watershed coefficients from regression equations of Narayana (1968), the equation (Figure 3.8)  $y = 0.775x + 0.125$  has been used. In this expression "x" represents the values of  $C_f$  computed in this study and "y" represents the parameter to be used in the regression equations of Narayana.

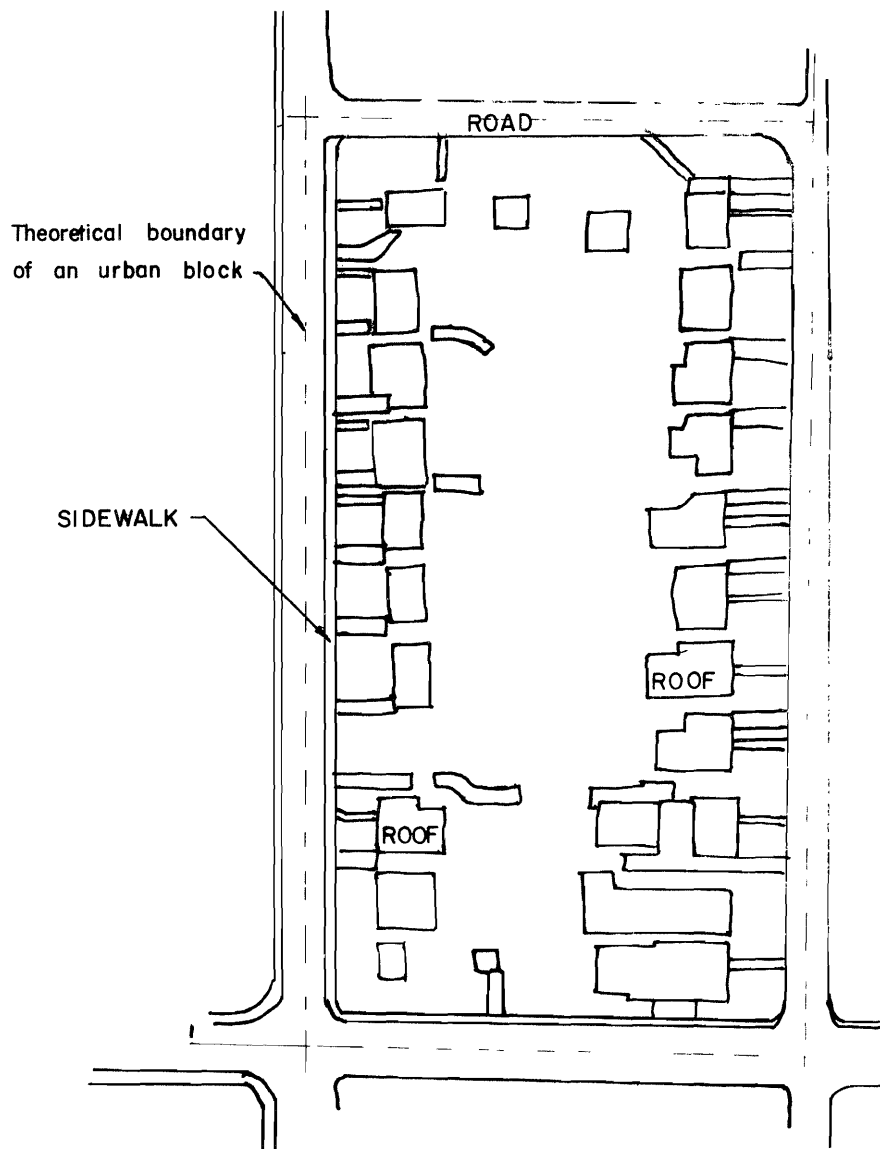


Figure 3.4. Typical urban residential block showing impervious areas.

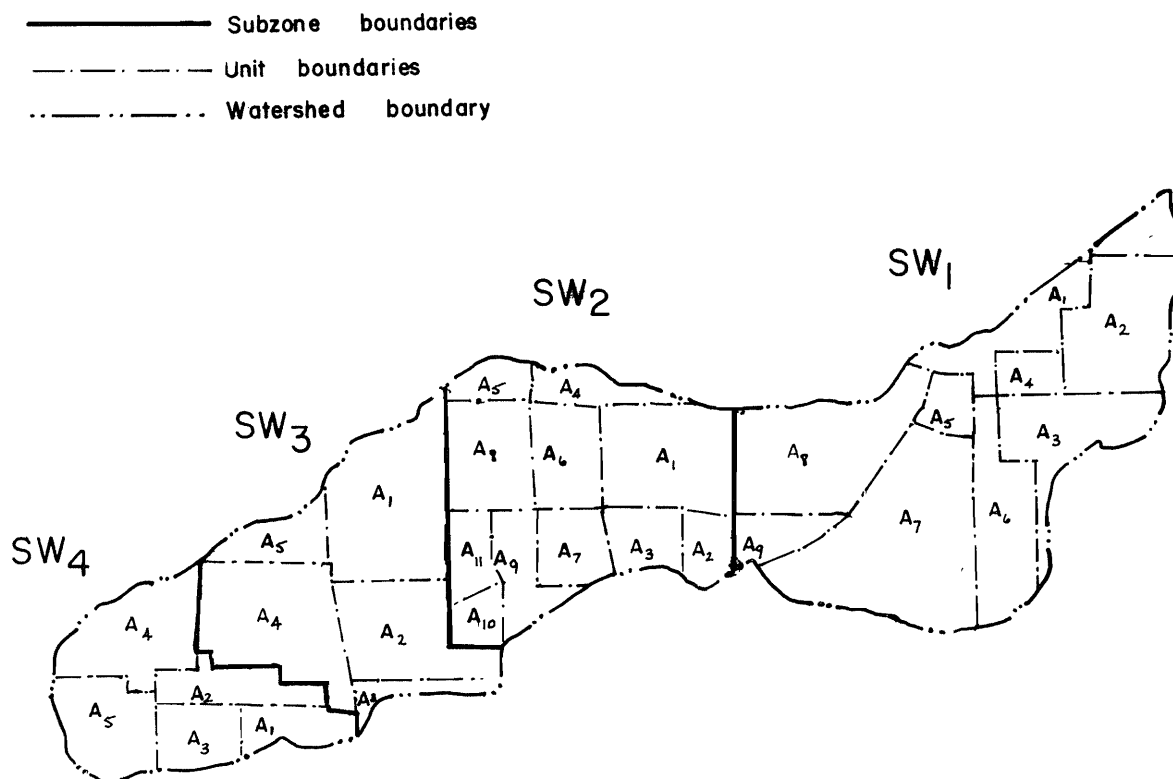


Figure 3.5. Subzone and unit boundaries of 38th Street Waller Creek Watershed in 1951.

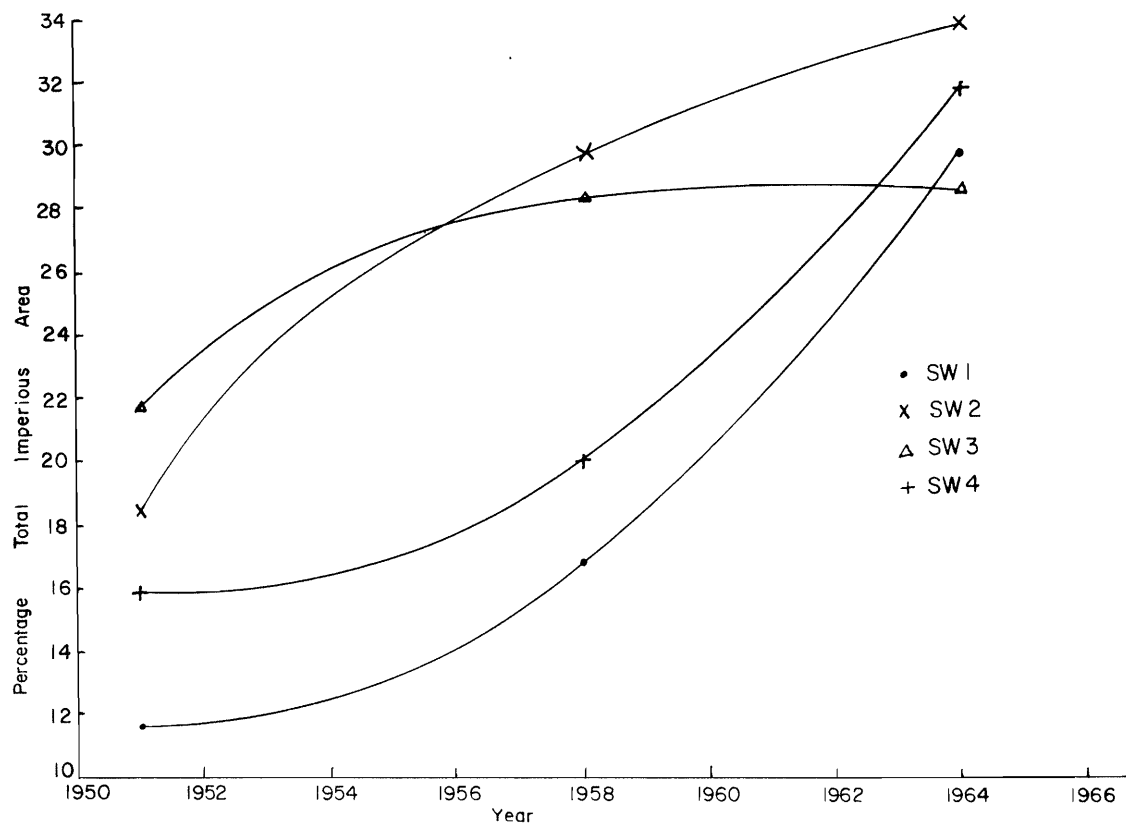


Figure 3.6. Yearly variation of percentage impervious cover in subwatersheds.

Table 3.1. Form of data collected on subwatersheds of Waller Creek  
(subwatershed SW<sub>1</sub>, 1951).

A <sub>i</sub>	Area (in. <sup>2</sup> )	Roads	Sidewalks	Parking Lots	Roofs	Total Paved	Total Non-paved	i <sub>i</sub> (in.)
A <sub>1</sub>	53.04	0	0	0	0	0	53.04	22.6
% <sub>1</sub>		0	0	0	0	0	100.0	
A <sub>2</sub>	36.17	3.36	.94	0	2.71	7.02	29.15	30.0
% <sub>2</sub>		9.3	2.6	0	7.5	19.4	80.6	
A <sub>3</sub>	36.32	0	0	0	0	0	36.32	21.7
% <sub>3</sub>		0	0	0	0	0	100.0	
A <sub>4</sub>	7.63	1.0	.08	.42	.25	1.75	5.88	20.7
% <sub>4</sub>		13.0	1.0	5.5	3.3	23.0	77.0	
A <sub>5</sub>	16.88	1.5	0	2.0	2.4	5.9	10.98	16.2
% <sub>5</sub>		8.9	0	11.9	14.2	35.0	65.0	
A <sub>6</sub>	17.42	1.62	.70	0	1.93	4.25	13.17	17.7
% <sub>6</sub>		9.3	4.0	0	11.1	24.4	75.6	
A <sub>7</sub>	35.52	3.1	.17	0	.48	3.75	31.77	13.8
% <sub>7</sub>		8.7	0.5	0	1.4	10.5	89.5	
A <sub>8</sub>	59.56	6.23	.63	0	1.8	8.66	50.9	9.9
% <sub>8</sub>		10.5	1.1	0	3.0	14.5	85.5	
A <sub>9</sub>	6.55	0	0	0	0	0	6.55	2.0
% <sub>9</sub>		0	0	0	0	0	100.0	
Total	269.09	16.81	2.52	2.42	9.57	31.33	237.76	
%		6.2	0.9	0.9	3.6	11.6	88.4	

L<sub>m</sub> = 17.72

L<sub>f</sub> = 0.535

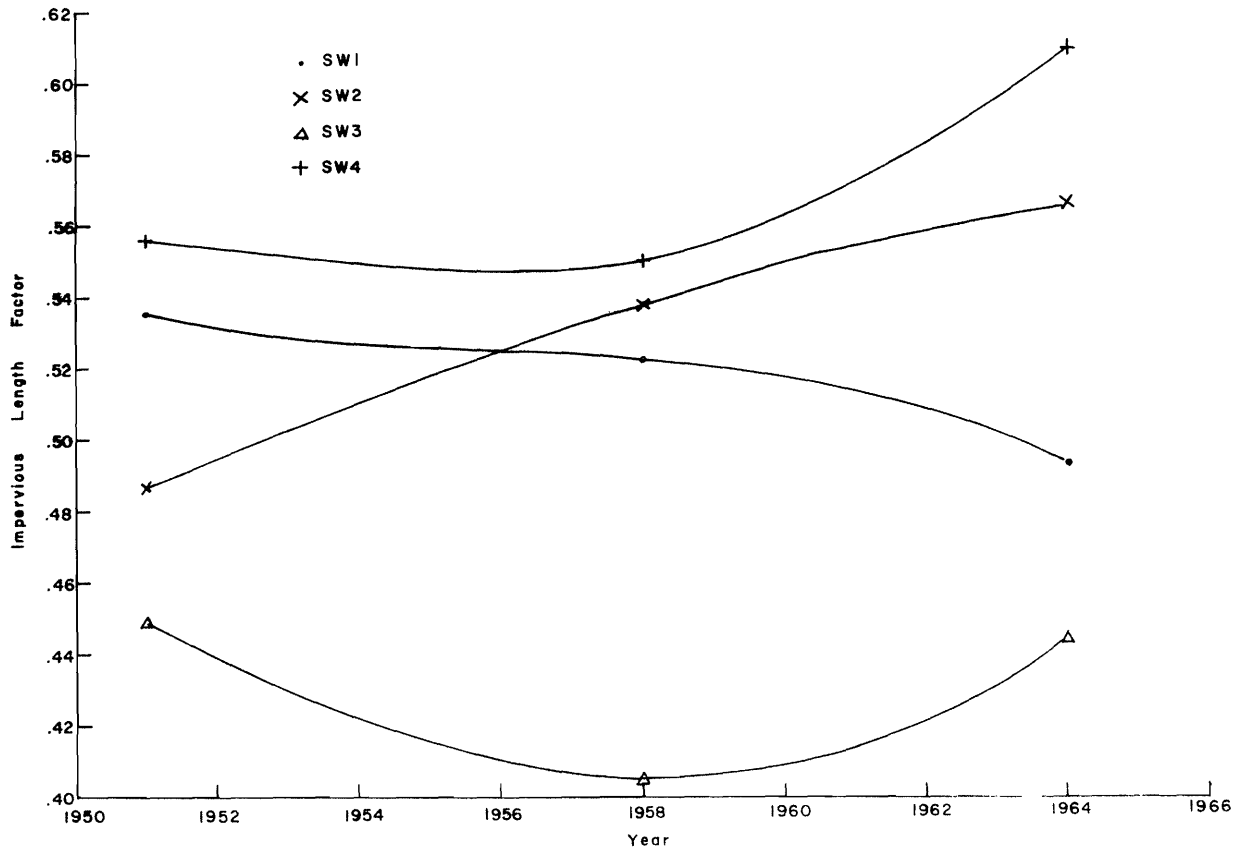


Figure 3.7. Yearly variation of characteristic impervious length factor in subwatersheds.



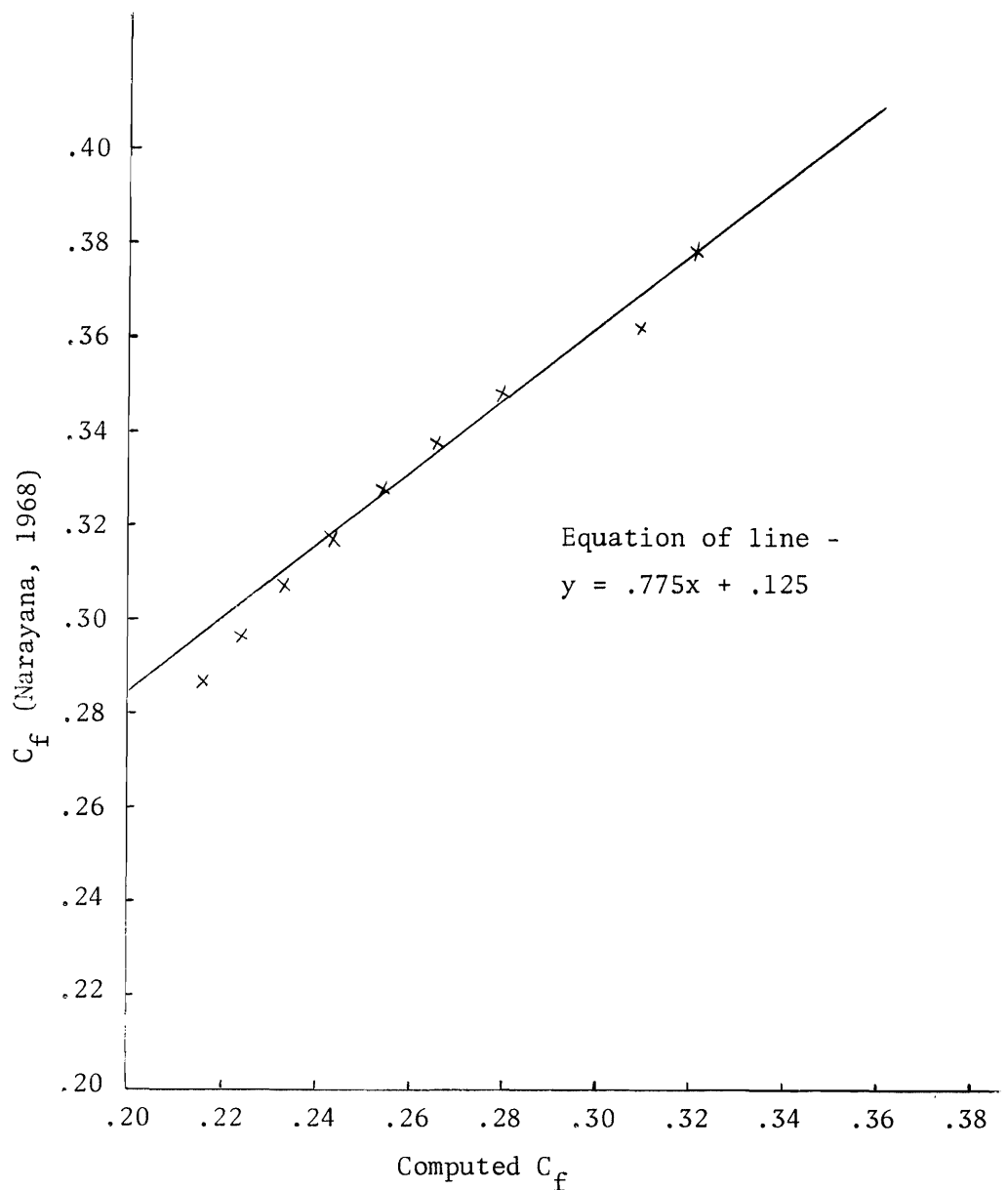


Figure 3.8. Graph of computed versus Narayana's percentage impervious cover.

Table 3.2. Comparison of urban parameters computed by Narayana (1969) and this writer.

Year	$C_f$	$L_f$	$C_f^{**}$	$L_f^{**}$
1956	.216	.461	.2865	.4679
1957	.224	.463	.2963	.4744
1958	.233	.469	.3070	.4818
1959	.243	.475	.3172	.4835
1960	.254	.480	.3275	.4861
1961	.266	.486	.3377	.4879
1962	.279	.490	.3480	.4905
1963	.293	.497		
1964	.308	.502	.3625	.4942
1965	.321	.505	.3787	.4948

\*\*Narayana (1969), p. 47, Table 6.2.

### Watershed Coefficients

The watershed coefficients of interception,  $S_i$ , maximum and minimum infiltration rates,  $f_o$  and  $f_c$  respectively, depression storage,  $S_d$ , and time of rise,  $t_R$ , describe the fundamental hydrologic behavior of a watershed. A sixth coefficient is the lag time,  $T_L$ , for each section of channel in the subwatershed model.

Narayana (1969) developed a series of regression equations which relate the watershed coefficients to the urban parameters. Over a ten year period 48 storms were simulated on the urban watershed model. The values of the watershed coefficients were adjusted for each storm until the best results were obtained between computed and observed storms. The order of importance of matching computed to observed storm hydrographs was as follows: (1) peak discharge, (2) rise time, (3) total volume of outflow, and (4) total duration of outflow.

Knowing both the watershed coefficients and the urban parameters for each year of study the following multiple regression equations were developed for the Waller Creek urban watershed:

$$S_i = -0.780 - 0.214C_f + 2.476L_f \quad (3.1)$$

$$f_o = 2.029 - 2.986C_f - 1.141L_f \quad (3.2)$$

$$f_c = 1.066 - 1.222C_f - 0.973L_f \quad (3.3)$$

$$S_d = -1.069 + 0.580C_f + 2.679L_f \quad (3.4)$$

$$t_R = 52.26 - 83.70C_f + 75.60L_f \quad (3.5)$$

Equations (3.1) through (3.4) will be utilized to determine  $S_i$ ,  $f_o$ ,  $f_c$ , and  $S_d$ ; however, the use of Equation (3.5) is inappropriate for determining the rise time in the subwatershed model. Clearly, the rise time is a function of the absolute aerial extent of the catchment area. In Equation (3.5),  $t_R$  is a function of a constant and two ratios which do not account for the length of travel of water draining from the catchment area. To provide an estimate of rise time, recourse is made to Espey's regression equation for the 30-minute unit hydrograph of urban watersheds in the Southwestern United States.

$$T_{RU} = 20.8 \phi L^{0.29} S^{-0.11} I^{-0.61} \quad (3.6)$$

in which

$T_{RU}$	=	rise time of the unit hydrograph for an urban watershed
$L$	=	channel length in feet
$S$	=	channel slope in ft/ft
$I$	=	percentage impervious cover
$\phi$	=	urban factor which is an index of channelization = 0.8 for Waller Creek watershed (Espey, Jr., Morgan, and Masch, 1965)

The actual computation of the watershed coefficients from the urban parameters was accomplished using a digital computer program in Appendix C. Table 3.3 summarizes the urban parameters and watershed coefficients used on Waller Creek watershed.

### Determination of Lag Time

Unfortunately, the analog computer capacity available for this study was not sufficient to generate the expression,  $Q^{-2/5}$  (Equation (2.19)). The reduction of  $T_L$  to a single constant value for each subzone and each individual storm will therefore be described in detail. The loss of accuracy resulting from this simplification will be shown to be negligible.

Substituting the known physical constants of Table 3.4 into Equations (2.17) and (2.19), the following values of  $T_L$  are obtained for subwatersheds 2, 3, and 4:

$$\text{for } SW_2 \text{ -- } T_L = 102 Q_2^{-2/5} \text{ minutes} \quad (3.7)$$

$$\text{for } SW_3 \text{ -- } T_L = 89.8 Q_3^{-2/5} \text{ minutes} \quad (3.8)$$

$$\text{for } SW_4 \text{ -- } T_L = 67.3 Q_4^{-2/5} \text{ minutes} \quad (3.9)$$

Notice that there exists no  $T_L$  for  $SW_1$ . This is because the rise time,  $t_R$ , alone accounts for all rainfall excess discharging from subwatershed 1.

By assuming that the outflow from each subzone outlet point is proportional to the area drained upstream, the lag time for each subwatershed can be expressed in terms of the discharge at the 38th Street gaging station. The only problem remaining is to determine a single representative final discharge for each particular storm.

The storms simulated are generally of short duration, and have a sharp peak response. Also, the greatest volume of outflow occurs during the

Table 3.3. Summary of urban parameters and watershed coefficients used on Waller Creek watershed.

Year	SW	C <sub>f</sub>	L <sub>f</sub>	S <sub>I</sub> (inches)	S <sub>d</sub> (inches)	f <sub>o</sub> (in/hr)	f <sub>c</sub> (in/hr)	t <sub>R</sub> (min)
1956	1	.237	.525	.469	.475	.721	.265	75.83
	2	.339	.525	.447	.534	.418	.141	42.52
	3	.341	.410	.162	.227	.542	.250	43.67
	4	.263	.547	.518	.549	.620	.212	50.01
1957	1	.244	.524	.465	.477	.701	.258	73.09
	2	.347	.532	.463	.558	.385	.124	41.52
	3	.344	.406	.152	.217	.542	.252	43.48
	4	.270	.548	.519	.556	.598	.203	48.53
1958	1	.254	.522	.458	.477	.674	.247	69.57
	2	.230	.538	.476	.578	.355	.109	40.66
	3	.344	.405	.149	.216	.539	.251	53.29
	4	.279	.550	.522	.566	.568	.190	46.72
1959	1	.266	.520	.451	.478	.641	.235	66.01
	2	.352	.544	.484	.593	.329	.096	39.93
	3	.345	.406	.151	.219	.535	.249	43.20
	4	.290	.555	.532	.586	.530	.172	44.82
1960	1	.280	.517	.440	.478	.603	.221	62.32
	2	.369	.550	.503	.618	.302	.081	39.31
	3	.345	.409	.159	.227	.532	.246	43.20
	4	.303	.562	.547	.612	.485	.150	42.89
1961	1	.296	.514	.429	.479	.560	.205	58.80
	2	.374	.554	.512	.632	.281	.070	38.78
	3	.346	.419	.171	.241	.524	.241	43.11
	4	.317	.572	.566	.645	.430	.123	40.85
1962	1	.313	.509	.413	.476	.515	.189	55.48
	2	.378	.558	.521	.645	.262	.061	38.35
	3	.346	.421	.188	.259	.516	.234	43.11
	4	.333	.583	.590	.683	.371	.093	38.96
1963	1	.333	.502	.392	.469	.463	.171	52.13
	2	.383	.562	.530	.659	.244	.051	37.92
	3	.347	.431	.213	.287	.502	.223	43.02
	4	.351	.595	.618	.729	.301	.058	36.98
1964	1	.355	.493	.365	.458	.406	.152	48.96
	2	.387	.565	.539	.672	.228	.042	37.58
	3	.347	.444	.245	.322	.487	.210	43.02
	4	.371	.609	.649	.778	.227	.020	35.17

Table 3.4. Physical characteristics of the subzones.

Subzone	Area (mi <sup>2</sup> )	Length of channel within subzone, d, (feet)	Width, b (feet)	Slope, S (ft/ft)	Manning's n
SW <sub>1</sub>	0.800				
SW <sub>2</sub>	0.618	5,000	25	0.0114	0.026
SW <sub>3</sub>	0.522	4675	15	0.00706	0.026
SW <sub>4</sub>	0.370	3465	20	0.0113	0.026

period bracketing the peak discharge. The peak rate of outflow has, therefore, been chosen as the representative final discharge.

The area computations which relate the 38th Street discharge to subzone outflows are given as follows:

$$\text{for } SW_2 \text{ -- } \frac{\text{Area of } SW_1 + SW_2/2}{\text{Area of } SW_1 + SW_2 + SW_3 + SW_4} = 0.433$$

$$\text{for } SW_3 \text{ -- } \frac{\text{Area of } SW_1 + SW_2 + SW_3/2}{\text{Area of } SW_1 + SW_2 + SW_3 + SW_4} = 0.727$$

$$\text{for } SW_4 \text{ -- } \frac{\text{Area of } SW_1 + SW_2 + SW_3 + SW_4/2}{\text{Area of } SW_1 + SW_2 + SW_3 + SW_4} = 0.920$$

The expressions for lag time, therefore, become:

$$\text{for } SW_2 \text{ -- } T_L = 143 Q_p^{-2/5} \quad . \quad . \quad (3.10)$$

$$\text{for } SW_3 \text{ -- } T_L = 102 Q_p^{-2/5} \quad . \quad . \quad (3.11)$$

$$\text{for } SW_4 \text{ -- } T_L = 69.6 Q_p^{-2/5} \quad . \quad . \quad (3.12)$$

Table 3.5 summarizes the lag time values obtained from substituting the known peak discharges for each storm into Equations (3.10) to (3.12). The use of known values of  $Q_p$  was felt justified

because (1) the approximation procedure was necessitated by lack of equipment, not mathematical theory, and (2) the very small range of lag time values for each subzone in light of the scale of 1" = 3 hours of the U.S. Geological Survey runoff data.

#### Precipitation Inputs

One of the primary advantages of the subzone approach to watershed modeling is the ability to provide for unique storm hyetographs for each subwatershed. Figure 3.9, illustrates the subzone boundaries, the rain gage network, and the Thiessen's polygon network for the watershed.

Second, the precipitation contributed by each Thiessen's polygon to each subzone in 30-minute intervals is calculated by the digital computer program in Appendix B.

#### Other Physical Parameters

In order to facilitate channel routing of subzone outflows, several additional physical characteristics of the watershed are necessary. These include the channel lengths between outflow points of subzones, Manning's  $n$  for each channel section, channel slope,  $S$ , and width,  $b$ , for each section. Notice that all of the additional parameters are describing the channel characteristics necessary for flood routing of flows.

Table 3.4 summarizes the average values of subwatershed channel characteristics for the years 1951 to 1965.

Table 3.5. Summary of the lag time values for each storm simulated on the Waller Creek watershed.

Storm Date	$Q_p$ (cfs)	Lag Time $T_L$ (min)		
		$SW_2$	$SW_3$	$SW_4$
06/12/57	500	11.9	8.5	5.8
10/13/57	370	13.4	9.5	6.5
10/13/57	518	11.8	8.4	5.7
10/15/57	323	14.2	10.1	6.9
04/26/58	465	13.7	9.8	6.7
07/06/58	500	11.9	8.5	5.8
04/08/59	228	16.3	10.6	7.9
09/23/59	468	13.7	9.8	6.7
10/16/60	230	16.3	10.6	7.9
10/28/60	1975	7.0	5.0	3.4
02/16/61	355	13.7	9.8	6.7
07/09/61	1380	8.0	5.7	3.9
06/10/62	1420	7.9	5.6	3.8
08/24/62	620	10.9	7.8	5.3
06/16/64	580	11.2	8.0	5.5
09/27/64	1340	8.0	5.7	3.9

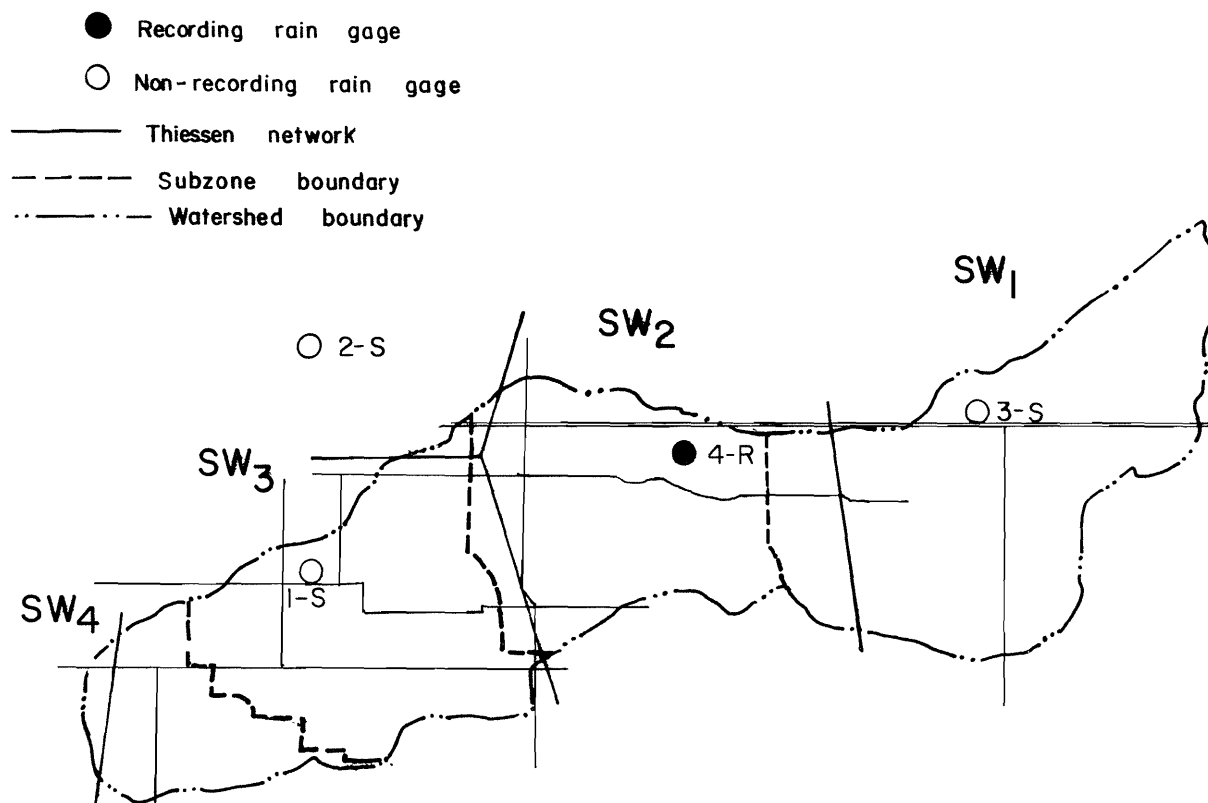


Figure 3.9. Relationship of Thiessen polygon network and subzones of the 38th Street Waller Creek watershed.

## Chapter IV

### ANALOG COMPUTER PROGRAMMING

Programming an analog computer consists of properly organizing a system of electrical components to simulate a set of mathematical expressions. The electronic analog must be scaled with respect to time and magnitude to make optimum use of the electrical model without exceeding its voltage limitations.

The basic mathematical operations which individual components perform are addition, multiplication, and integration. Proper arrangement of components permits a very wide range of functions and equations to be simulated. Voltage is the dependent variable with all voltages as functions of the independent variable time.

The analog program illustrated in Figure 4.1 represents the runoff process of the four subwatersheds in the Waller Creek model. The program was spread out over three patch panel stations on the analog computer at the Utah Water Research Laboratory. Station 3 was utilized as the control station with the other stations as slaves. The small numbered (x)'s in the diagram indicate trunk lines connecting two stations. Subwatershed 1 is all that is necessary to model an entire watershed at one time. Six sections of the program will be discussed as follows:

1. Precipitation inputs;
2. Interception;
3. Infiltration;
4. Depression storage;
5. Overland flow; and
6. Channel routing.

#### Precipitation

The precipitation input to the model is in the form of a voltage step function. The amount of voltage for each step is determined from the known precipitation in inches as calculated by the digital computer program in Appendix B and the magnitude scale factor.

The input device consists of servo-driven potentiometric function generators. After setting the desired step voltage inputs on a series of potentiometers, activation of the automatic relay mechanism switches a mechanical wiper from one potentiometer to another in one second intervals, thereby producing the proper input.

#### Interception

The expression for capacity interception rate is given by the equation:

$$i_{cc} = i e^{-P/S_I} \quad . \quad . \quad . \quad . \quad . \quad . \quad (4.1)$$

with

$$i_{ca} = i \text{ for } i < i_{cc}$$

and

$$i_{ca} = i_{cc} \text{ for } i > i_{cc} \quad . \quad . \quad . \quad (4.2)$$

The computer program for generating Equation (4.1), and the conditions defined by Equation (4.2), are represented in Figure 4.2.

In order to generate the function  $100 e^{-P/S_I}$  the following procedure is adopted. Let

$$y = 100 e^{-P/S_I}$$

$$\frac{dy}{dt} = - \frac{1}{S_I} \frac{dp}{dt} e^{-P/S_I} \times 100$$

$$\frac{dy}{dt} = - \frac{i}{S_I} y \quad . \quad . \quad . \quad . \quad (4.3)$$

Referring to Figure 4.2, this circuit generates the desired result.

$$i_1 = i \left( 1 - e^{-P/S_I} \right) \quad . \quad . \quad . \quad (4.4)$$

The diode output is always positive, which assures that Equation (4.2) is satisfied.

The initial condition on integrator 1 can be set to either -100 volts to represent a dry watershed or to 0 volts to represent a recent storm which has satisfied the interception storage. A potentiometer may be introduced between a -100 volt source and the initial condition on integrator 1. This would enable any intermediate value of capacity interception rate to be set in order to properly represent antecedent conditions.

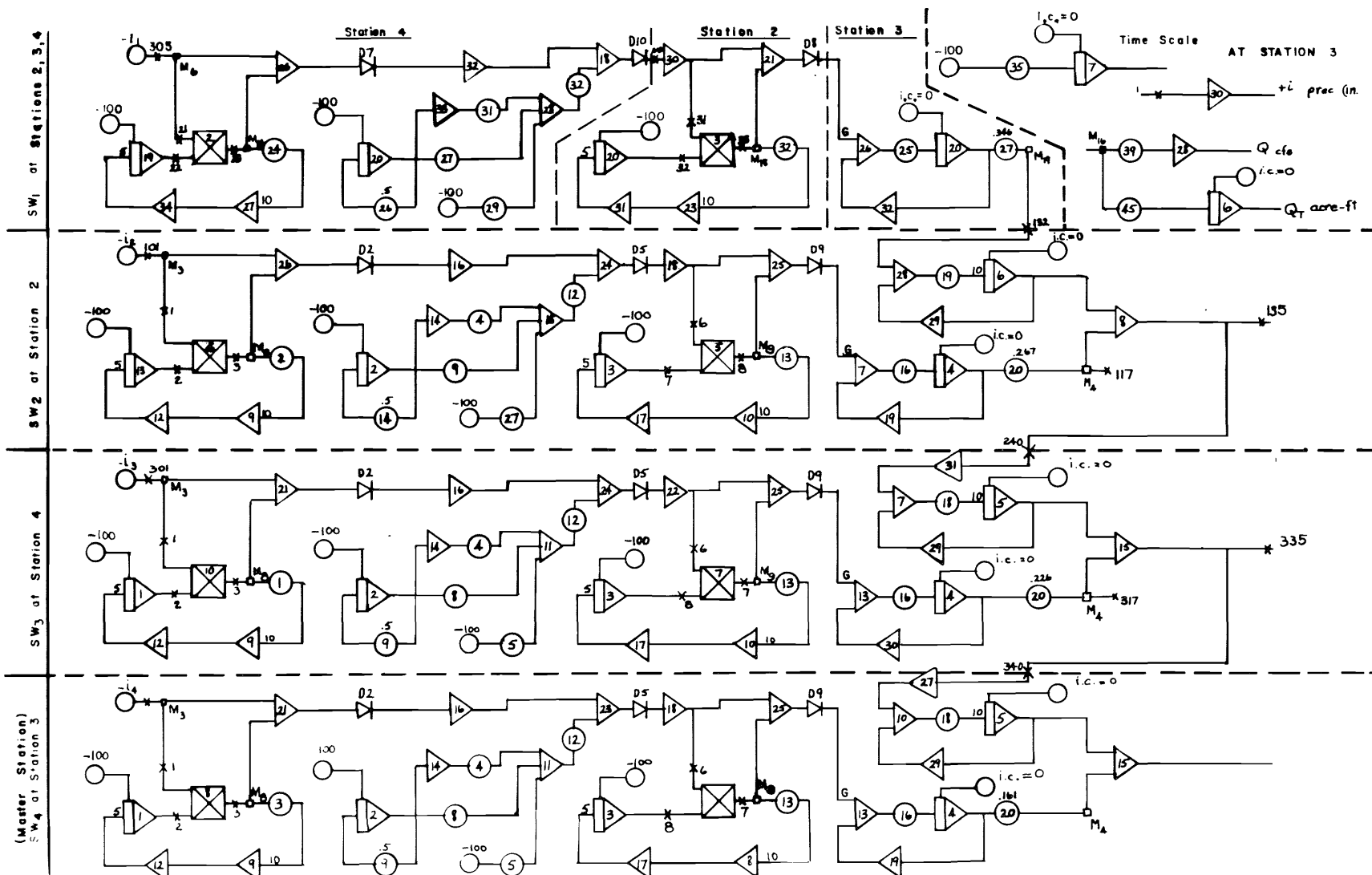


Figure 4.1. The subwatershed analog computer program.

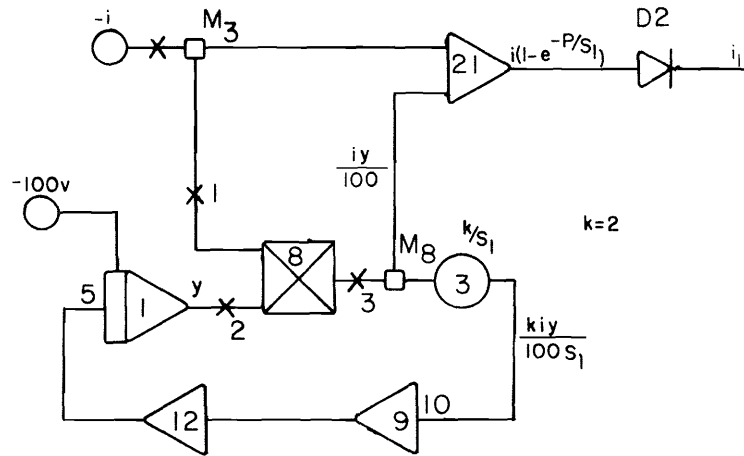


Figure 4.2. Analog circuit for generating the expression for interception rate.

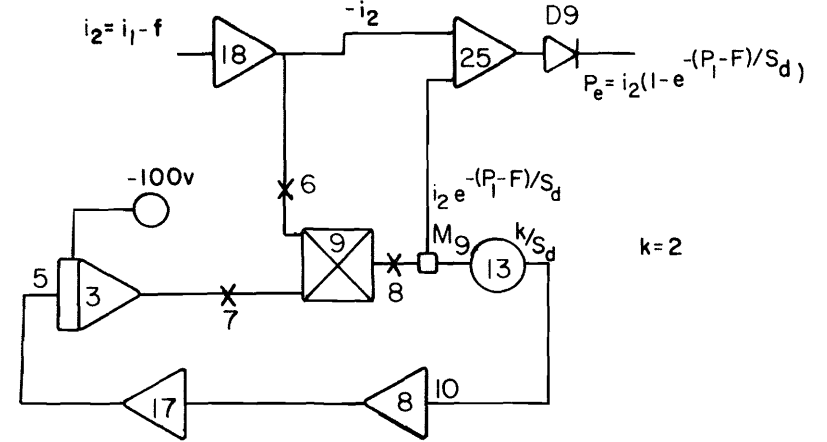


Figure 4.4. Analog circuit for generating the expression for inflow rate into depression storage.

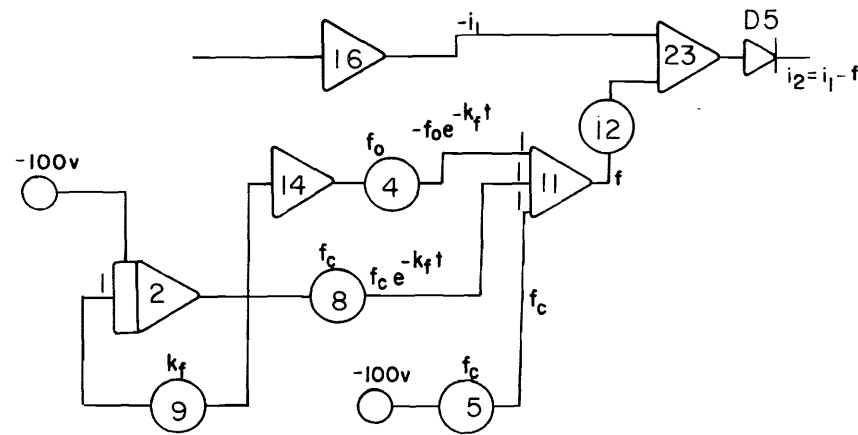


Figure 4.3. Analog circuit for generating the expression for infiltration rate.

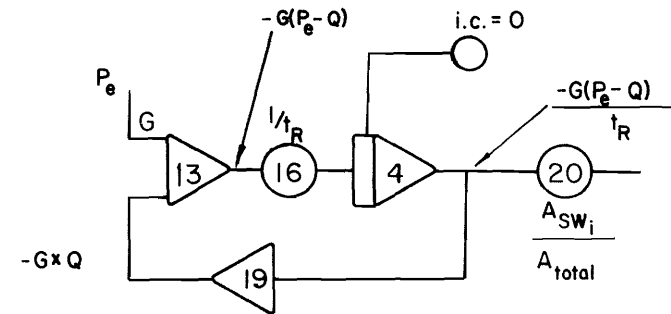


Figure 4.5. Analog circuit for obtaining the subwatershed outflow hydrograph.



### Infiltration

The equation for infiltration capacity rate is given by:

$$f = f_c + (f_o - f_c) e^{-k_f/t} \quad . \quad . \quad . \quad (4.5)$$

with

$$f_a = i_1 \text{ for } i_1 < f$$

and

$$f_a = f \text{ for } i_1 > f \quad . \quad . \quad . \quad (4.6)$$

Equation (4.5) along with the conditions defined by Equation (4.6), are represented by Figure 4.3.

The output of amplifier 11 is  $f$ . Diode 5 provides for the boundary conditions given by Equation (4.6).

### Depression Storage

The equations for rate of inflow into depression storage is given by:

$$\sigma_c = i_2 e^{-(P_1 - F)/S_d} \quad . \quad . \quad . \quad (4.7)$$

with

$$\sigma_a = i_2 \text{ when } \sigma_c > i_2$$

and

$$\sigma_a = \sigma_c \text{ when } \sigma_c < i_2 \quad . \quad . \quad . \quad (4.8)$$

The computer program for these expressions is given by Figure 4.4. This circuit is similar to Figure 4.2, since Equations (4.7) and (4.1) are similar.

### Overland Flow Routing

The expression that governs the routing of rainfall excess is given by:

$$p_e - Q = t_R \frac{dQ}{dt} \quad . \quad . \quad . \quad (4.9)$$

or

$$\frac{dQ}{dt} = (p_e - Q)/t_R \quad . \quad . \quad . \quad (4.10)$$

The circuit diagram for Equation (4.10) is shown by Figure 4.5. A possible source of overloading is amplifier 13. The gain (G) is adjusted to values of 1, 5, or 10 to produce the highest voltage output without overloading the amplifier.

Potentiometer 20 is equal to the area of the respective subwatershed divided by the total areal extent of the watershed. If the catchment area being modeled is an entire watershed, then the potentiometer is set at 1.0.

### Channel Flow Routing

The expression developed for channel flow routing is given as follows:

$$Q_i - Q_o - T_L \frac{dQ_o}{dt} \quad . \quad . \quad . \quad (4.11)$$

or

$$\frac{dQ_o}{dt} = \frac{1}{T_L} (Q_i - Q_o) \quad . \quad . \quad . \quad (4.12)$$

The analog circuit for Equation (4.12) is shown in Figure 4.6.

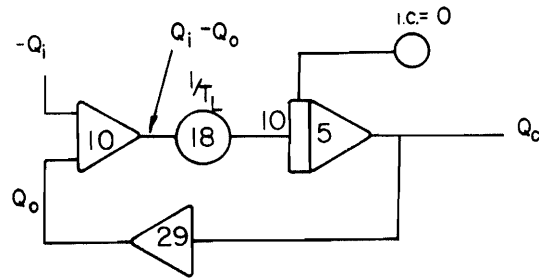


Figure 4.6. Analog circuit for channel routing.

### Time Scaling

A time scale of 1 second of computer time equal to 30-minutes of prototype time reflects the choice made by Narayana (1969). Further, the various statistical equations for the unit hydrograph characteristics of this watershed were developed for 30-minute durations (Espey, Jr., Morgan, Masch, 1965). This time scale of  $\Delta t$  equal to 30-minutes permits the direct use of the rise time of Espey's unit hydrograph as the characteristic time ( $t_R$ ) in the routing of precipitation excess ( $p_e$ ).

### Amplitude Scaling

The choice of a proper amplitude scale factor for a problem is as important as the choice of time scale. Ideally a problem should be scaled so as to

keep the solution output voltage as high as possible without exceeding the maximum voltage range of the computer ( $\pm 100$  volts). Low output voltages may be significantly affected by the inherent noise level of the computer.

From a study of the precipitation data from the Waller Creek watershed, the maximum value of precipitation occurring in any 30-minute interval has been found to be 2.26 inches (occurred on May 16, 1965). In order to keep the peak voltage of the input close to 100 volts, a scale of 1 inch equal to 40 volts is selected.

#### Final Outflow and Total Volume of Outflow

The final outflow from the computed storm is properly scaled for input to an X-Y plotter through the use of the circuit shown in Figure 4.7. The total volume of outflow is read directly from a voltmeter connected across integrator 6.

Station 3 containing subwatershed 4 and part of subwatershed 1 is the master control station. Therefore, all trunk line connections used for the purpose of determining an output are connected to station 3.

The circuit in Figure 4.7, when properly scaled, is used to determine the discharge and total volume from any of the following combinations of subwatersheds:

$SW_1$  only -- at  $M_{19}$

$SW_2$  only -- at trunk 117

$SW_3$  only -- at trunk 317

$SW_4$  only -- at  $M_4$

$SW_1 + SW_2$  -- at trunk 135

$SW_1 + SW_2 + SW_3$  -- at trunk 335

$SW_1 + SW_2 + SW_3 + SW_4$  -- at

amplifier 15

In order to obtain the output of any single subwatershed, a change of sign should be made before using the output scaling circuit shown in Figure 4.7.

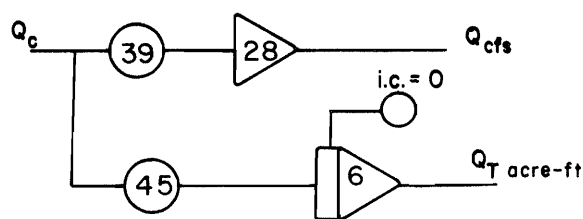


Figure 4.7. Analog circuit for scaling the final outflow and total volume of outflow.

The following expression illustrates the conversion of computer output in volts ( $Q_c$ ) to discharge in cubic feet per second:

$$Q_{cfs} = \frac{Q_c (\text{volts/sec}) \times A (\text{mi}^2) \times 5280 \times 5280 (\text{ft}^2/\text{mi}^2)}{G \times 40 (\text{volt/in}) \times 12 (\text{in/ft}) \times 30 (\text{min/sec}) \times 60 (\text{sec/min})}$$

for  $G = 5$

$$Q_{cfs} = Q_x \times A \times 6.454$$

$G$  refers to the gain on amplifiers in the overland and channel routing circuit section shown in Figure 4.5.

To determine the total volume of outflow in acre-feet, the following conversion is used:

$$Q_T = \frac{Q_{ct} (\text{volts}) \times A (\text{mi}^2) \times 640 (\text{acre/mi}^2)}{G \times 40 (\text{colts/inch}) \times 12 (\text{inch/ft})}$$

for  $G = 5$

$$Q_T = Q_{ct} \times A \times .2666$$

When using the subzone model the total area of the watershed is used in Equations (4.12) and (4.13) since the corresponding fraction of the watershed area is placed on potentiometer 27 of station 3 and potentiometers 20 of stations 2, 3, and 4.

Table 4.1 summarizes the potentiometer settings in the model. The potentiometer numbers refer to the pots used in Figures 4.2 through 4.7, which are also the numbers of the components in subwatershed 4.

\* Refers to seconds of computer time.

Table 4.1. Summary of the potentiometer settings for computer variables.

Program section	Pot Number	Potentiometer setting for the analog computer
Interception	3	$P_3 = k/(S_I \times 40)$ $S_I$ in inches $k = 2$
Infiltration	9	$P_9 = k_f$ (Equation (4.5)) assumed equal to 0.5 in this study
	5	$P_5 = f_c \times 20$ $f_c$ in inches/hour
	4	$P_4 = f_c \times 20$ $f_c$ in inches/hour
	8	$P_8 = f_o \times 20$ $f_o$ in inches/hour
	12	$P_{12}$ -- Higher accuracy may be achieved by multiplying the setting of pots 4, 5, and 8 by a factor greater than 1 and then correspondingly reducing the final infiltration value, $f$ , by the reciprocal of that factor.
Depression Storage	13	$P_{13} = k/(S_d \times 40)$ $S_d$ in inches $k = 2$
Overland-flow routing	16	$P_{16} = 30/t_R$ $t_R$ in minutes
	20	$P_{20} = \text{area of subzone/area of entire watershed}$
Channel routing	18	$P_{18} = 3.0/T_L$ $T_L$ in minutes

## MODEL APPLICATION

The results presented in this chapter determine the usefulness of the subwatershed model in predicting the outflow hydrograph from points within a watershed. The outcome of applying the regression equations of Waller Creek watershed to the rural Wilbarger Creek watershed are also presented.

### Waller Creek Watershed

A comparison of the observed and simulated final outflow hydrograph characteristics for 16 storm events on the 38th Street Waller Creek watershed is presented in Table 5.1. The hydrograph characteristics considered were peak discharge,  $Q_p$ , rise time,  $t_R$ , total volume of outflow,  $Q_T$ , and storm duration,  $t_D$ .

The storms selected for simulation satisfied two general characteristics. First, the total accumulated precipitation was sufficient to satisfy all abstractive losses so that excess rainfall was available to supply runoff. Secondly, most of the storms are typically convective, producing the bulk of their total precipitation in a short time period. Because the watershed model does not account for that portion of precipitation which is not faced with abstraction by interception, infiltration, or depression storage, the analog model simulates more accurately storms in which losses are satisfied quickly. Precipitation falling on a road, parking lot, or roof surface which is connected directly to the drainage system of the basin suffers little, if any, abstractive losses.

To explain further, a long duration storm, of relatively low but steady rainfall intensity, may not produce any outflow hydrograph at all in the analog model. This result was observed in a number of cases where the minimum infiltration rate was large enough to absorb the low rate of rainfall and the depression storage was capable of absorbing any sudden bursts of higher intensity rainfall. Clearly, a portion of the incoming precipitation falls on impervious areas which produce a certain amount of rainfall excess even for very small rainfall intensities. Figure 5.1 illustrates the two extremes of storm type and the reaction of the analog model. In summary, although all abstractive losses must be satisfied before commencement of runoff in the computer model, this is not true of the real runoff process in urban watersheds.

In Table 5.2 the results of linear regression analysis between observed and computed hydrograph characteristics is given.

Examination of the  $R^2$  values for each hydrograph characteristic indicates a good degree of correlation. To test the hypothesis that the regression equations provide a significant relationship between observed and computed results, the t-test is applied. The smallest value of  $R$  and  $R^2$ , from the total outflow duration,  $t_D$ , are utilized as follows:

$$t \left( 1 - \frac{\alpha}{2}, n-2 \right) = \frac{r \sqrt{n-2}}{\sqrt{1-r^2}}$$

$$t \left( 1 - \frac{\alpha}{2}, 14 \right) = \frac{.788 \sqrt{16-2}}{\sqrt{1-.622}} = 4.66$$

from a table of the t-distribution

$$t(.9995, 14) = 4.140$$

Obviously, a high degree of significance exists for all the derived regression relations. Figures 5.2 through 5.5 illustrate the scatter points about the 45 degree line for the tabulated values of hydrograph characteristics. Figure 5.6 shows several examples of simulated and actual final outflow hydrographs.

The reason for extending the use of the urban watershed model to a subzone level is the spatial flexibility obtained in inputting and outputting information from the watershed. The specific capabilities of the subwatershed model are:

1. An individual rainfall hyetograph may be input to each subwatershed, allowing one to take full advantage of data from any existing raingage network.
2. The watershed coefficients can be adjusted to reflect the degree of urbanization within each subwatershed.
3. The contribution of each subwatershed to the final outflow hydrograph can be obtained.
4. The outflow hydrograph at each subwatershed discharge point can be determined.

The inherent flexibility of the subwatershed model is demonstrated for the storm of June 16, 1964, in the following series of figures.

Figure 5.7 shows the hydrographs generated at the outflow point of each subzone as if each were a completely independent subwatershed of its own.

In Figure 5.8 the simulated hydrographs at each subzone outflow location are shown superimposed on one another. In this instance  $SW_1$  refers

Table 5.1. Comparison of observed and simulated hydrograph characteristics at Waller Creek watershed.

Storm Date	Q <sub>pc</sub> cfs	Q <sub>po</sub> cfs	t <sub>RC</sub> min	t <sub>RO</sub> min	Q <sub>TC</sub> ac-ft	Q <sub>TO</sub> ac-ft	t <sub>DC</sub> min	t <sub>DO</sub> min
06/12/57	519	500	115	105	103	115	600	660
10/13/57	402	370	342	312	85	136	720	720
10/13/57	502	518	240	240	115	210	720	720
10/15/57	490	323	58	75	132	240	840	1100
04/26/58	692	465	81	78	79	69	510	580
07/06/58	360	500	60	30	37	45	360	540
04/08/59	236	228	96	84	32	42	492	650
09/23/59	505	468	132	90	61	58	498	605
10/16/60	222	230	162	180	30	38	660	700
10/28/60	1920	1975	384	480	380	502	900	1260
02/16/61	207	355	84	106	42	93	600	1080
07/09/61	1300	1380	326	300	220	182	780	1440
06/10/62	888	1420	126	198	108	147	360	480
08/24/62	650	620	71	92	112	99	540	744
06/16/64	922	580	240	270	144	123	600	720
09/27/64	1120	1340	204	210	180	175	540	600

Table 5.2. Regression between observed and simulated hydrograph characteristics, Waller Creek subwatershed model.

No.	Parameter	Number of Observations	Regression Equations	R	R <sup>2</sup>
1	a    Q <sub>p</sub>	16	Q <sub>pc</sub> = 108.3 + 0.8164 Q <sub>po</sub>	0.928	0.861
	b    Q <sub>p</sub>	16	Q <sub>pc</sub> = 63.55 + 0.8800 Q <sub>po</sub>	0.963	0.928
2	a    t <sub>R</sub>	16	t <sub>RC</sub> = 17.95 + 0.8540 t <sub>Ro</sub>	0.953	0.909
	b    t <sub>R</sub>	16	t <sub>RC</sub> = 10.49 + 0.8959 t <sub>Ro</sub>	0.976	0.953
3	a    Q <sub>T</sub>	16	Q <sub>TC</sub> = 14.77 + 0.7140 Q <sub>To</sub>	0.919	0.845
	b    Q <sub>T</sub>	16	Q <sub>TC</sub> = 5.829 + 0.7769 Q <sub>To</sub>	0.959	0.919
4	a    t <sub>D</sub>	16	t <sub>DC</sub> = 259.4 + 0.4421 t <sub>Do</sub>	0.788	0.622
	b    t <sub>D</sub>	16	t <sub>DC</sub> = 166.0 + 0.5607 t <sub>Do</sub>	0.888	0.788

a - method of least squares.

b - minimizing the square of normal deviations.

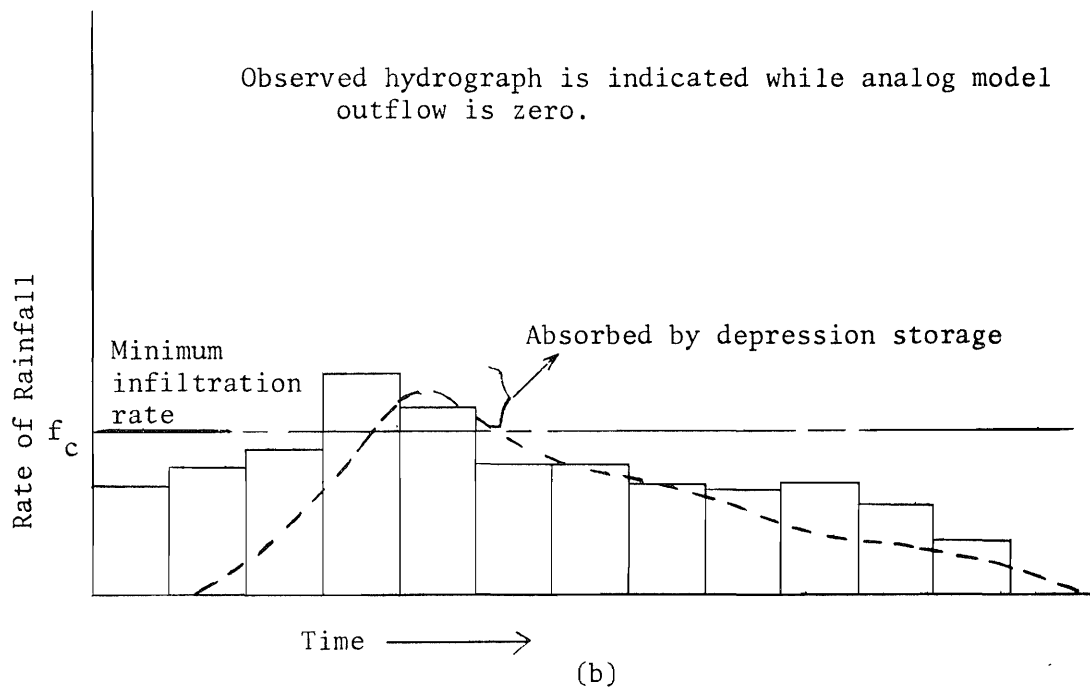
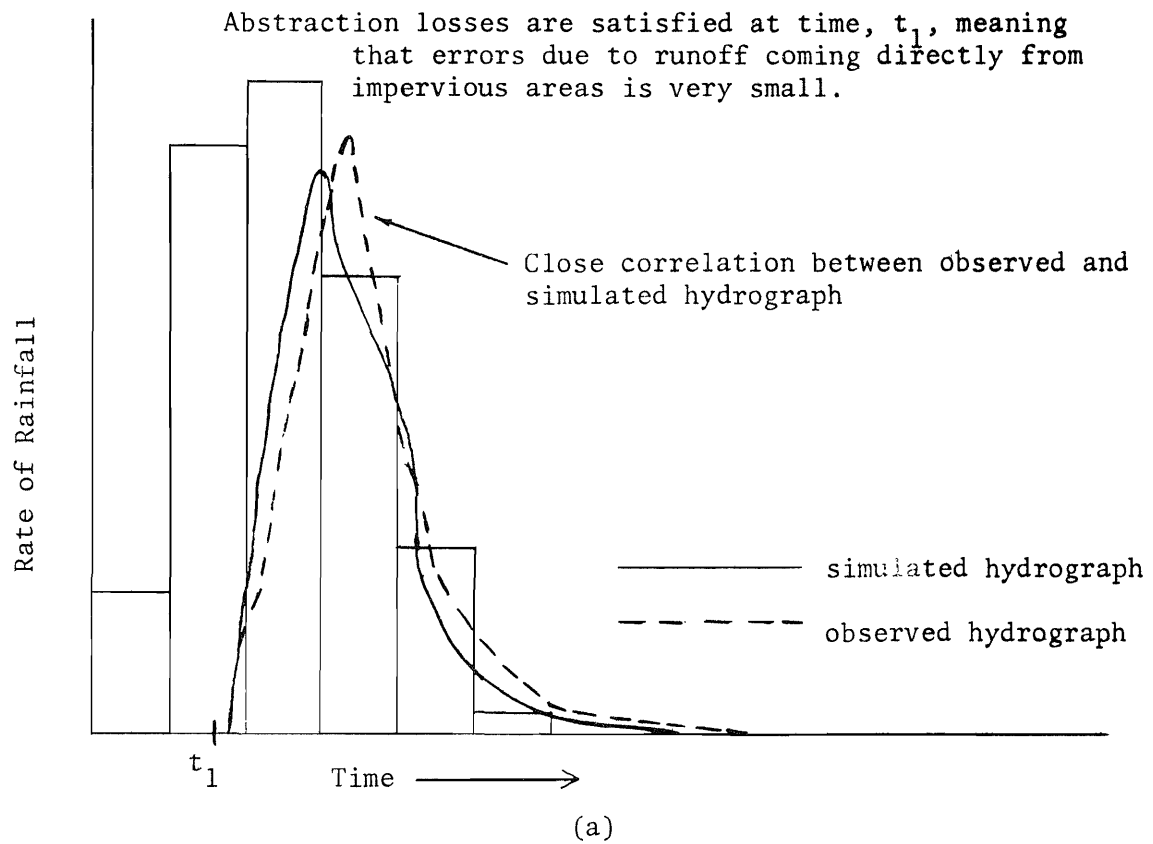


Figure 5.1. Comparison of observed and simulated responses to two extremes of storm input.

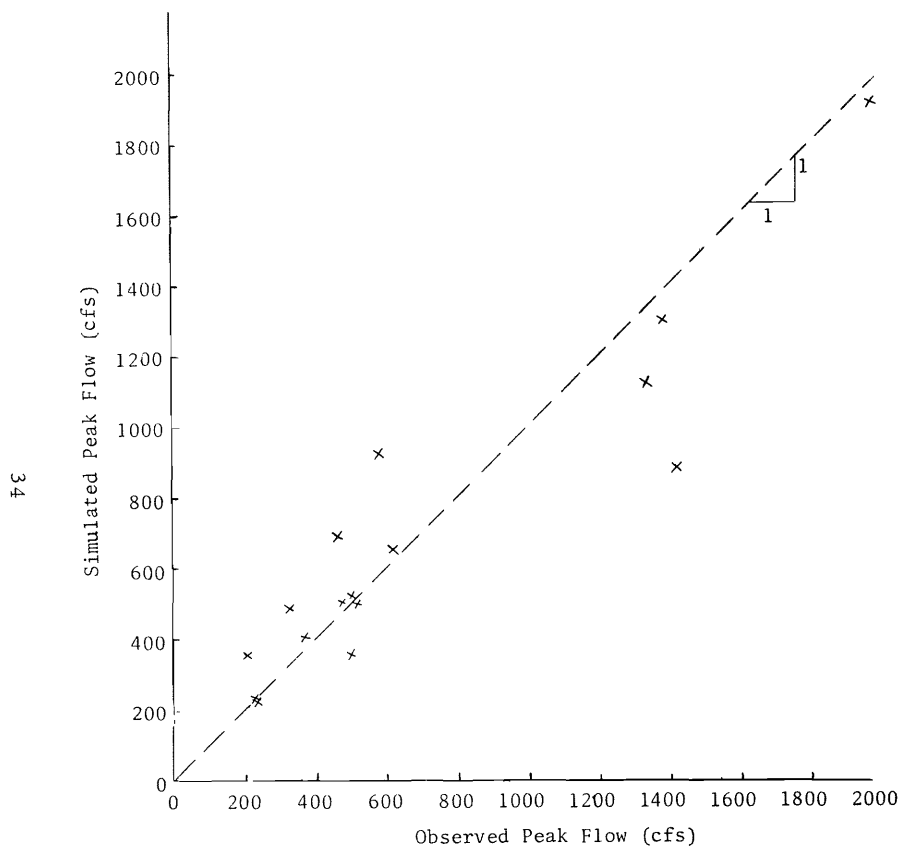


Figure 5.2. Observed versus simulated peak discharge for the Waller Creek subwatershed model.

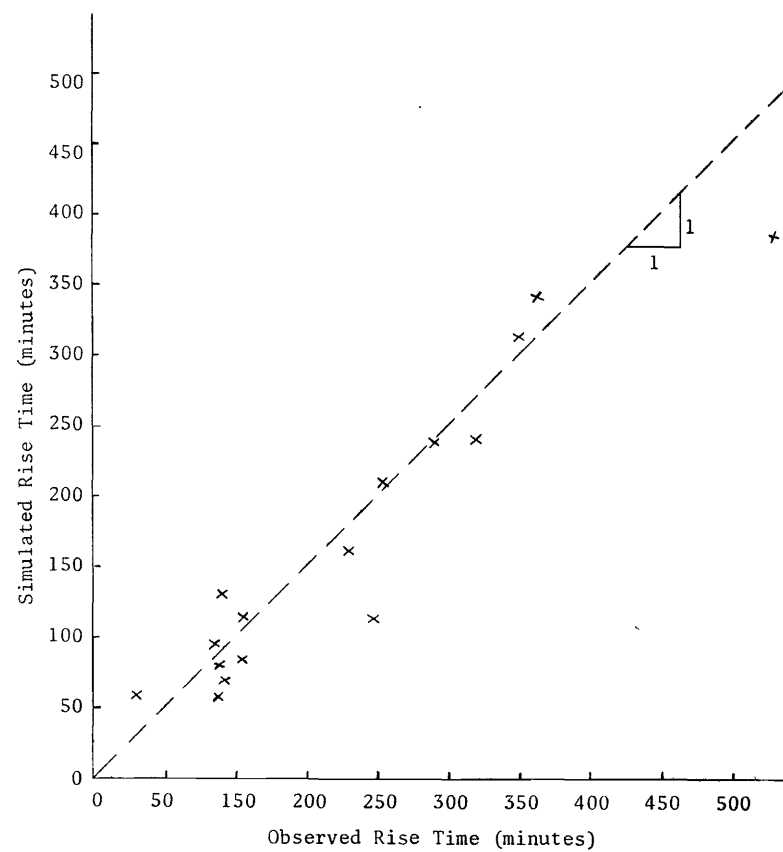


Figure 5.3. Observed versus simulated rise time for the Waller Creek subwatershed model.

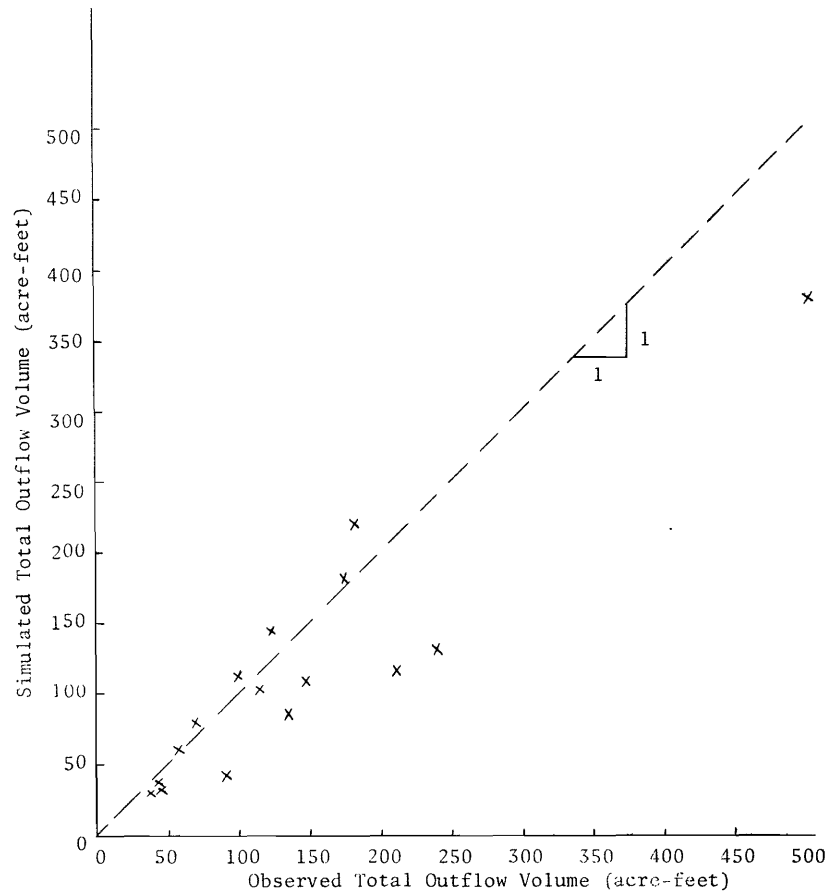
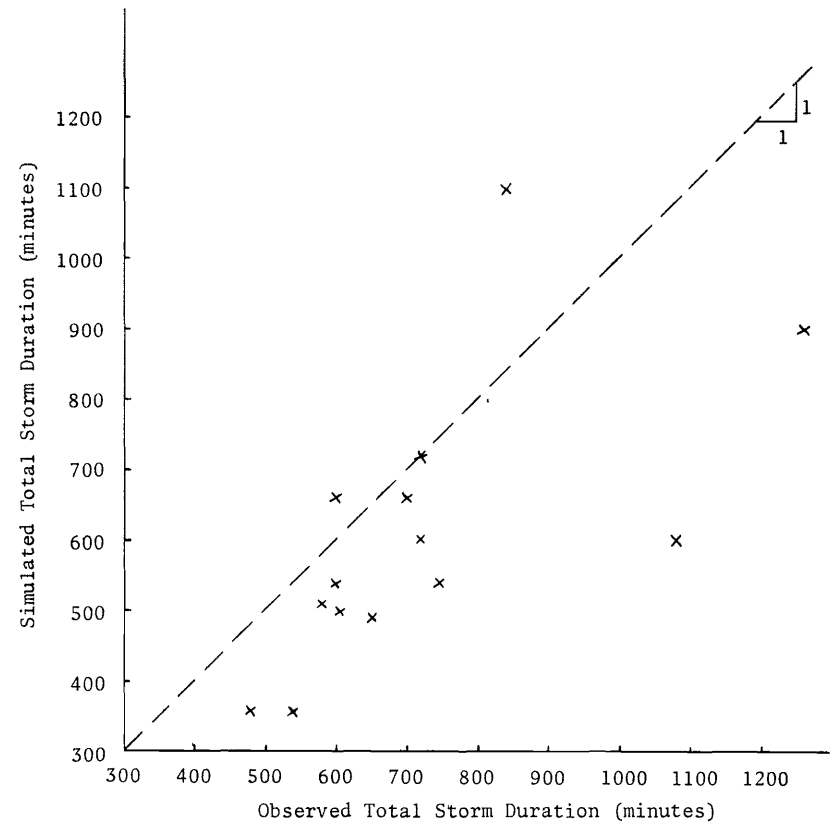


Figure 5.4. Comparison of observed and simulated total outflow volume for the Waller Creek subwatershed model.





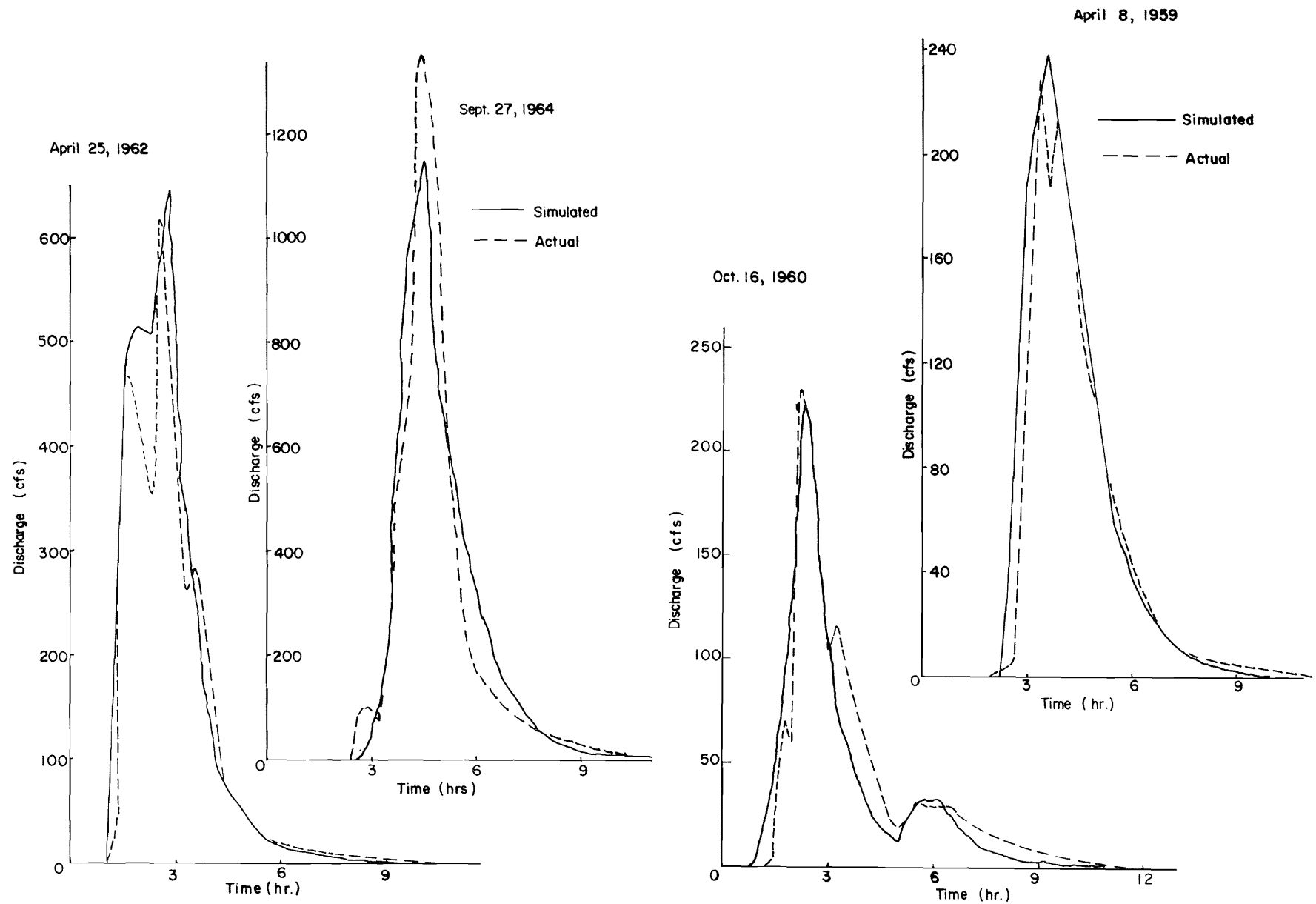


Figure 5.6. Comparison of observed and simulated final outflow hydrographs at the 38th Street gaging station of Waller Creek basin.

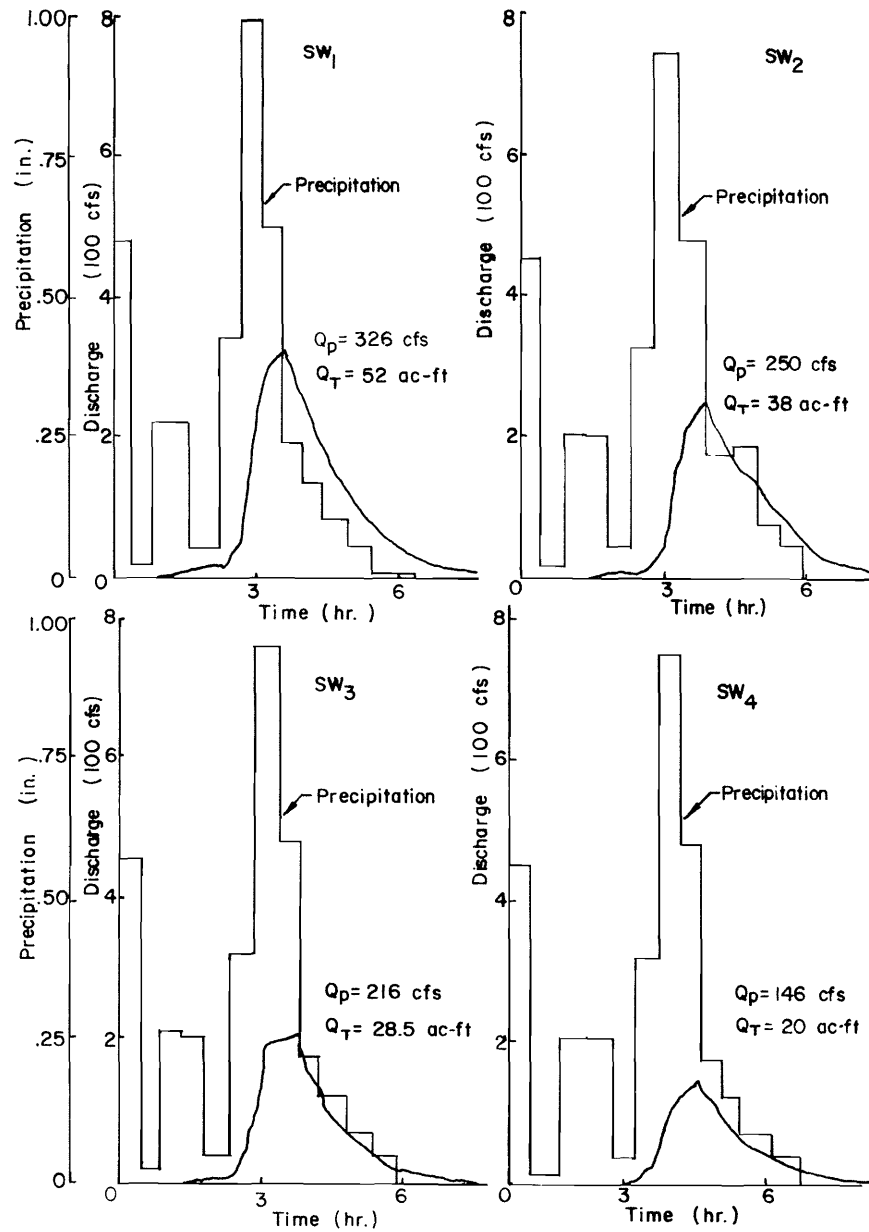


Figure 5.7. Individual subzone hydrographs for the storm of June 16, 1964 on Waller Creek watershed.

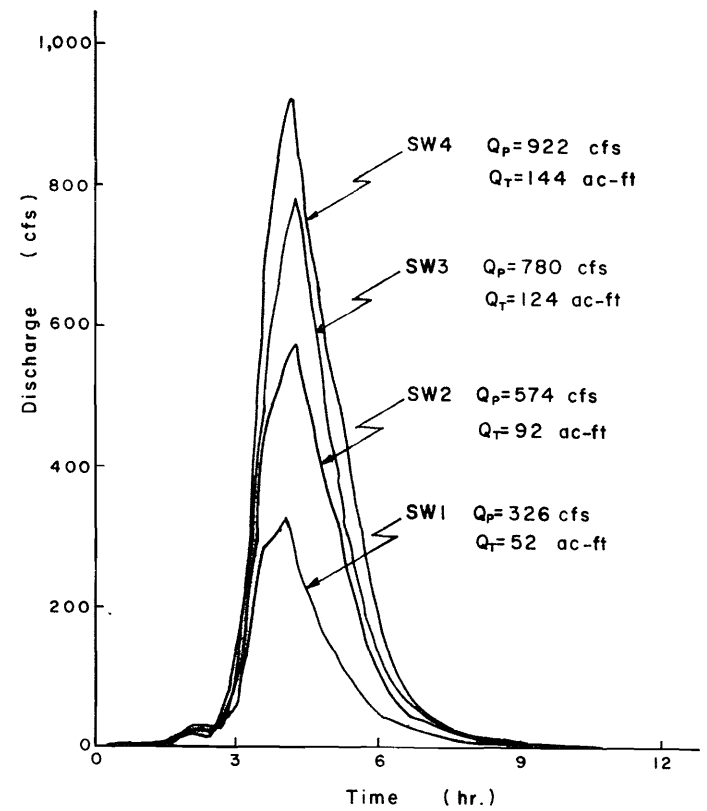


Figure 5.8. Outflow hydrographs at the end of each subwatershed for the storm of June 16, 1964.

to the discharge at the outlet of  $SW_1$ . At each outflow point the appropriate upstream hydrographs of Figure 5.7 are routed together.

Figure 5.9 illustrates the relative time lag between subzone outflow hydrographs recorded at their point of origin and at the 38th Street gaging station.

The hydrographs at the final outflow measuring point for all combinations of subwatershed outflow are illustrated in Figure 5.10.

A limited sensitivity analysis was performed in Figure 5.11. The effect of varying the minimum infiltration rate,  $f$ , in  $SW_1$  is shown for both the outflow of  $SW_1$  and the final outflow hydrograph.

The facility with which the subwatershed model handles problems in which spatially varied inputs and outputs are involved make it a useful tool in several important areas.

1. The study of the effects of storm characteristics on outflow hydrographs.
2. The design of detention structures at various locations within the watershed.
3. The design of a drainage system, particularly if the subzones are chosen small and numerous.
4. The prediction of the effects of land use (urbanization) on drainage outflow from a basin, through the use of the urban parameters of characteristic impervious length factor and percentage impervious cover.
5. Frequency analysis to determine the occurrence of specific peak outflows.

#### Wilbarger Creek Watershed

Located north of Pflugerville, and 13 miles north of Austin, Texas, Wilbarger Creek is similar in climate and geology to Waller Creek watershed (Figure 3.1). The headwaters are near the boundaries of Williamson and Travis counties (Figure 5.12), and about 3 miles upstream from the stream gaging station. The creek flows in a southeasterly direction to the Colorado River.

The principal land use of this rural watershed is farming and ranching. Runoff characteristics are slightly affected by contour farming practices and numerous stock tanks. In addition, vegetational cover, both pasture and agricultural, affect runoff characteristics in different seasons. It is not anticipated that land use will change.

Average slope of the channel is about 46 feet per mile. Soil is predominantly of variable thickness, and is underlain by Austin chalk which crops out at many places in the channel.

Instruments to collect rainfall and runoff data consist of three recording rain gages and a continuous recording stream-gaging station. Records at this station began August 9, 1963 (U.S. Geological Survey, 1966).

Wilbarger Creek watershed is being used to test the applicability of the Waller Creek regression equations relating urban parameters and watershed coefficients simply because no data are available on Waller Creek prior to urbanization. The purpose of this portion of the study is to test the applicability of the derived urban regression equations to the limiting case where there is almost no impervious cover. If the relationships provide good results on a rural watershed, then the regression equations are usable over a wide range of percentage impervious cover values.

The impervious cover of Wilbarger Creek watershed was assumed equal to zero. The characteristic impervious length factor was determined to be 0.587 from measuring the drainage from the center of the watershed to the final outflow point and dividing by the longest length of travel on the watershed. Table 5.3 summarizes the values of urban parameters and watershed coefficients utilized on Wilbarger Creek watershed.

Because there is no urbanization, the urban parameters and thus the watershed coefficients remain unchanged with time.

The results of utilizing the regression equations developed on the Waller Creek watershed for the rural Wilbarger Creek watershed are presented in Table 5.3. A comparison of two of the simulated and observed hydrographs is presented in Figure 5.13. A general discussion of the results will be presented in that a statistical analysis with only four observations has little meaning.

The simulated hydrographs provided reasonable results, although somewhat disappointing when compared to the results obtained from Waller Creek watershed. The variations between simulated and observed peak discharge and total volume of outflow seem to indicate that important factors affecting the runoff process have not been taken into account.

The fact that the regression equations used to determine the watershed coefficients were derived on Waller Creek watershed, means that account has not been taken of the full effect of seasonal changes of vegetational cover on the abstraction processes. The shape and topography of the

Wilbarger Creek watershed would likewise not be taken into consideration by the regression equations used. Most importantly, one should be careful in applying regression equations to a problem in which the independent variables are out of the range of those originally derived for the equations. In this case the smallest value of percentage impervious cover used in developing the regression equations was 28 percent. The value taken for percentage impervious cover in this study was 0 percent.

Finally, storm characteristics were not taken into consideration as was done in the subwatershed model. Spatial variations in rainfall intensity could easily produce the variations in hydrograph characteristics observed in any single instance.

More years of rainfall-runoff data will be required to make an adequate study of the runoff process from Wilbarger Creek watershed.

Table 5.3. Urban parameters and watershed coefficients utilized on Wilbarger Creek watershed.

$L_f = 0.587$	and	$C_f = 0$
$S_I = 0.670$ inches		
$f_o = 1.359$ inches/hour		
$f_c = 0.495$ inches/hour		
$S_s = 0.504$ inches		
$t_R = 99.7$ minutes		

Table 5.4. Comparison of observed and simulated hydrograph characteristics at Wilbarger Creek watershed.

Storm Date	$Q_{pc}$ cfs	$Q_{po}$ cfs	$t_{RC}$ min	$t_{RO}$ min	$Q_{TC}$ ac-ft	$Q_{TO}$ ac-ft	$t_{DC}$ min	$t_{DO}$ min
4/26/64	385	570	102	105	73	99	780	780
6/15/64	1320	870	528	588	220	119	600	600
6/16/64	1150	1760	426	360	260	468	960	1020
9/27/64	1100	1100	135	120	228	204	720	900

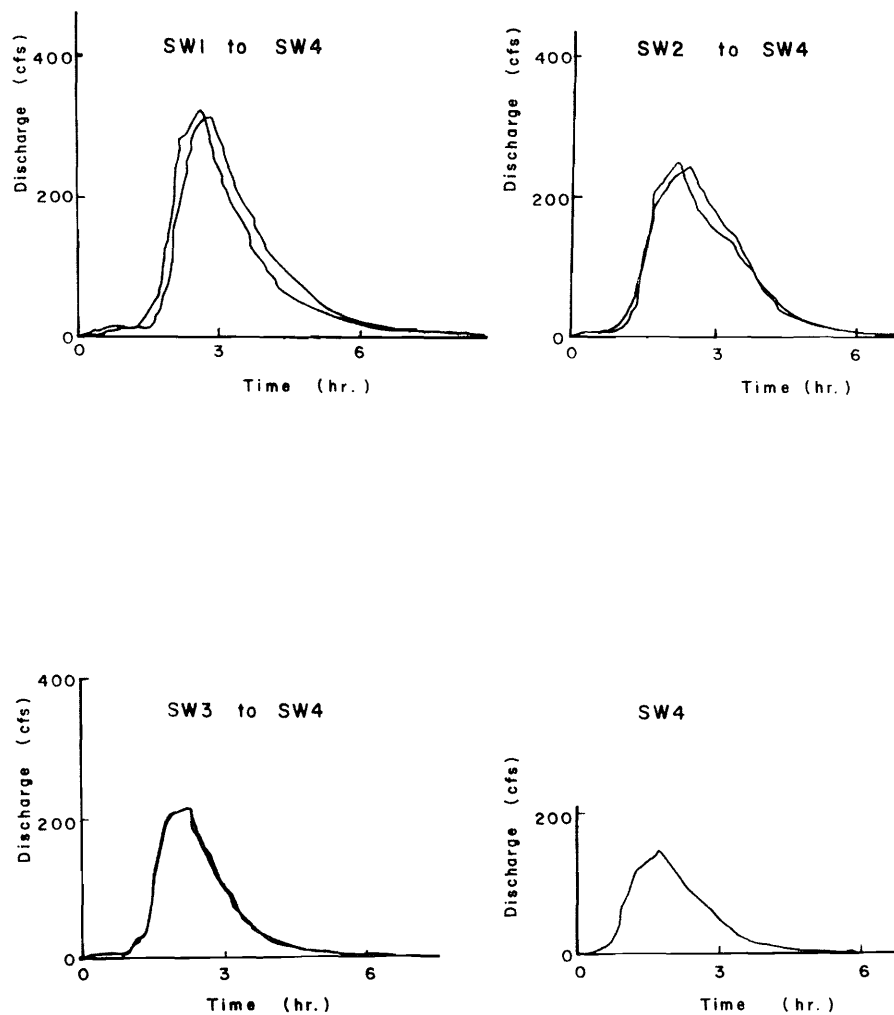


Figure 5.9. Effect of channel routing on outflow hydrographs for the storm of June 16, 1964.

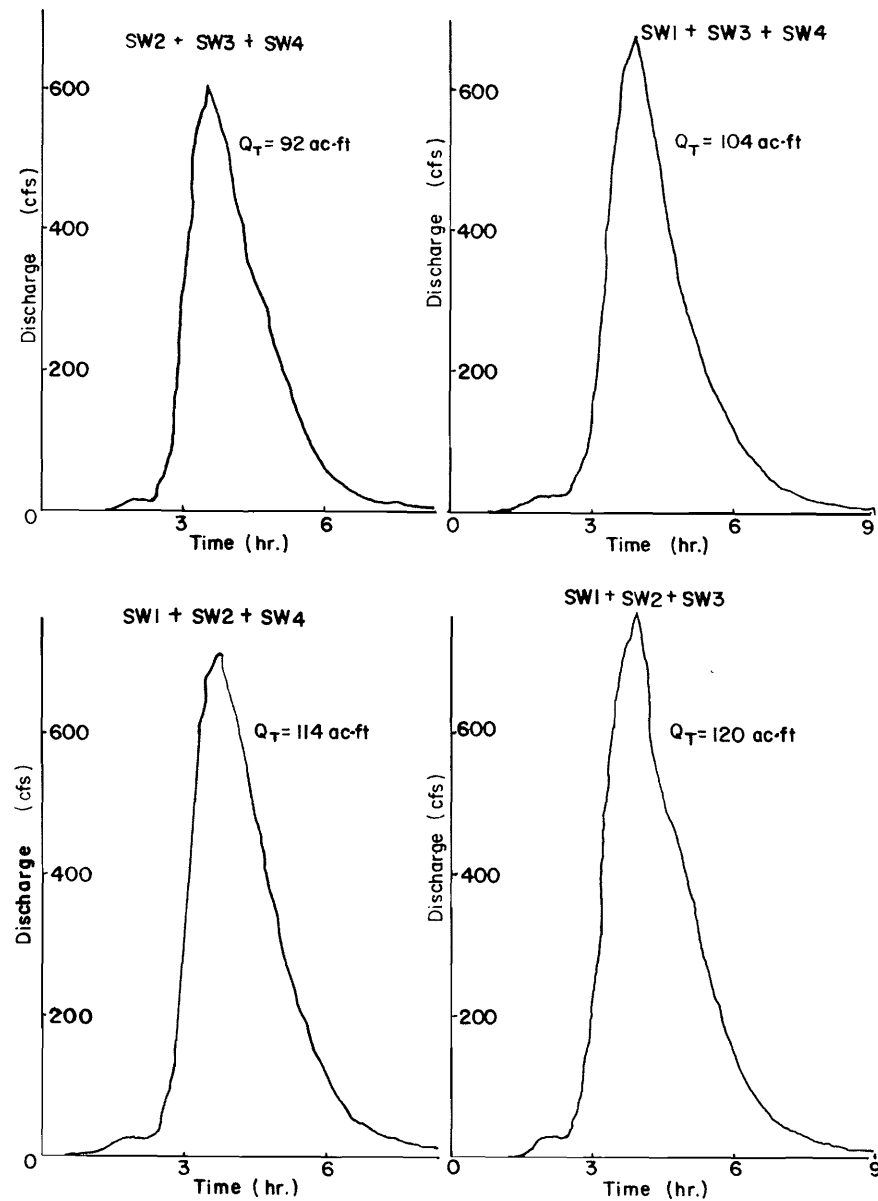


Figure 5.10. All combinations of subzone outflows simulated at the 38th Street gaging station for June 16, 1964 storm.

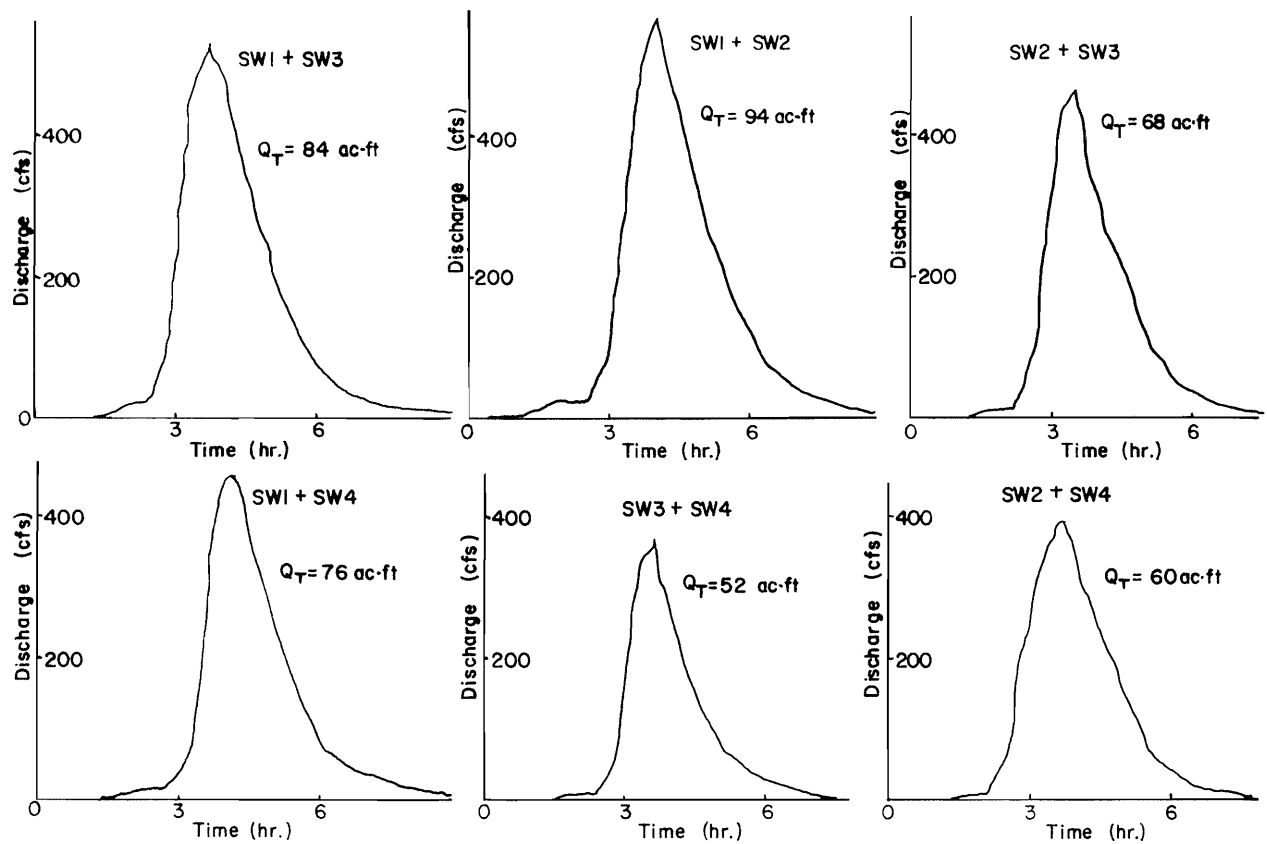


Figure 5.10. (Continued).

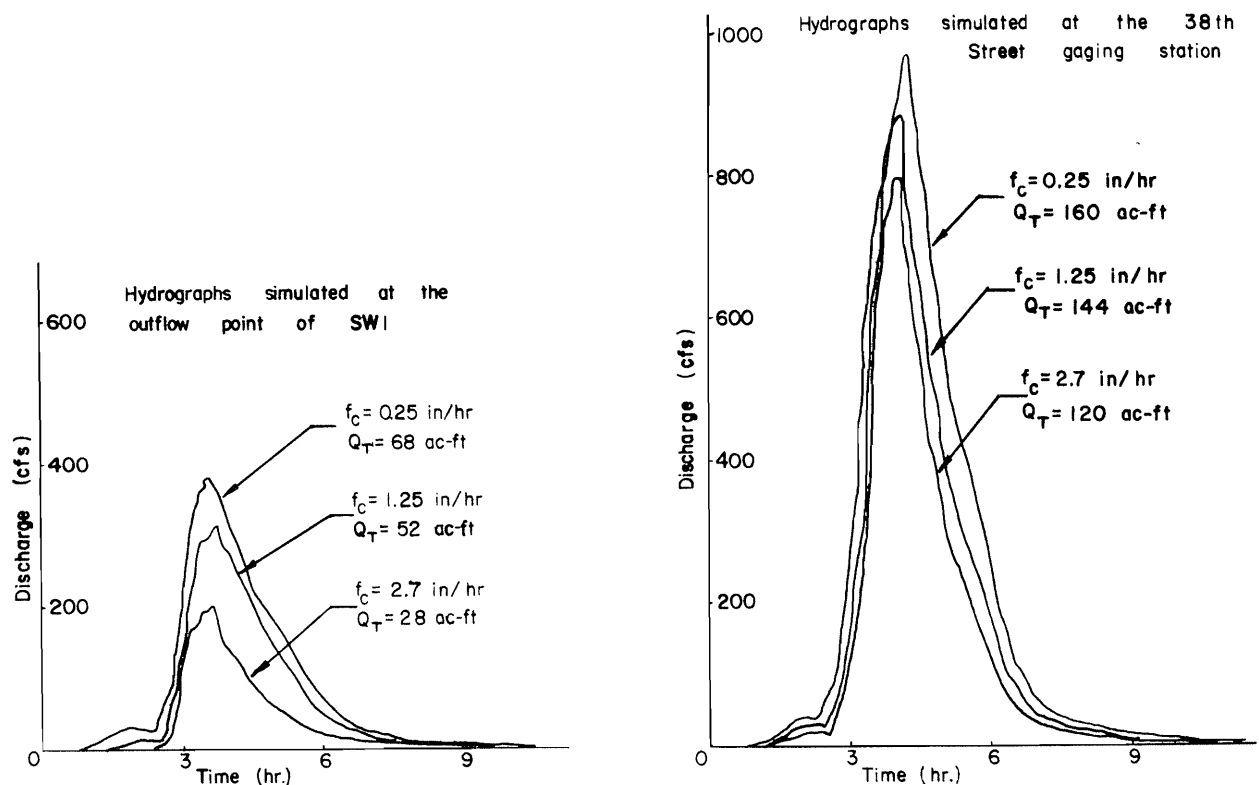


Figure 5.11. Sensitivity analysis limited to varying the minimum infiltration rate in  $SW_1$  for the storm of June 16, 1964 on Waller Creek basin.

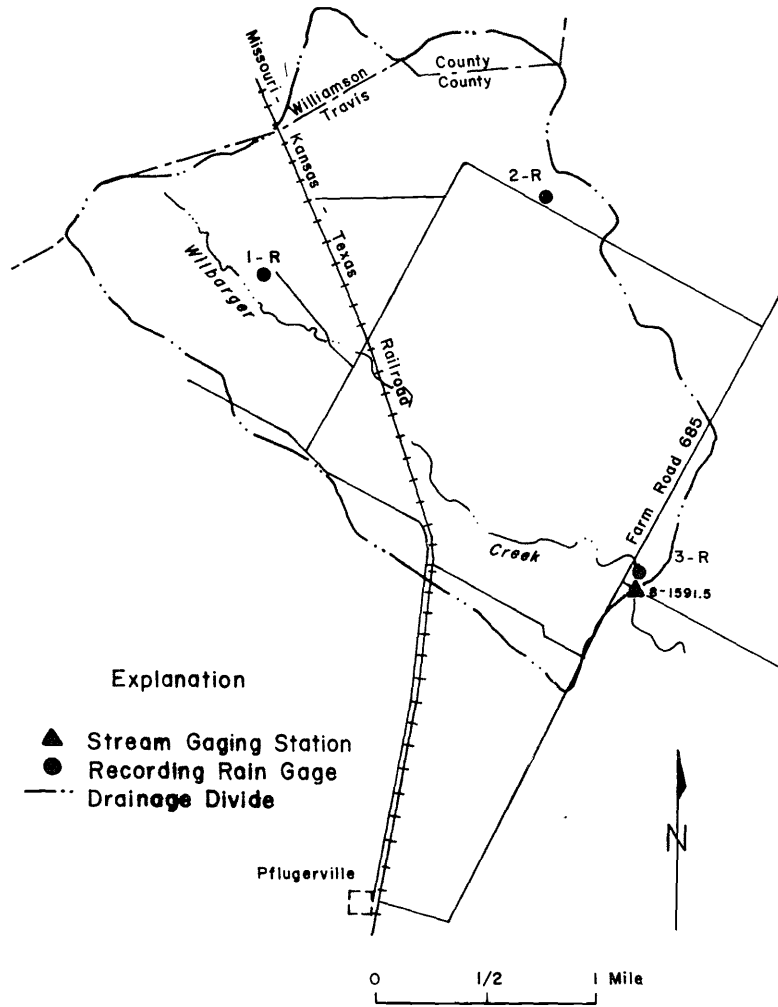


Figure 5.12. Map of Wilbarger Creek study area showing the locations of hydrologic instrument installations.

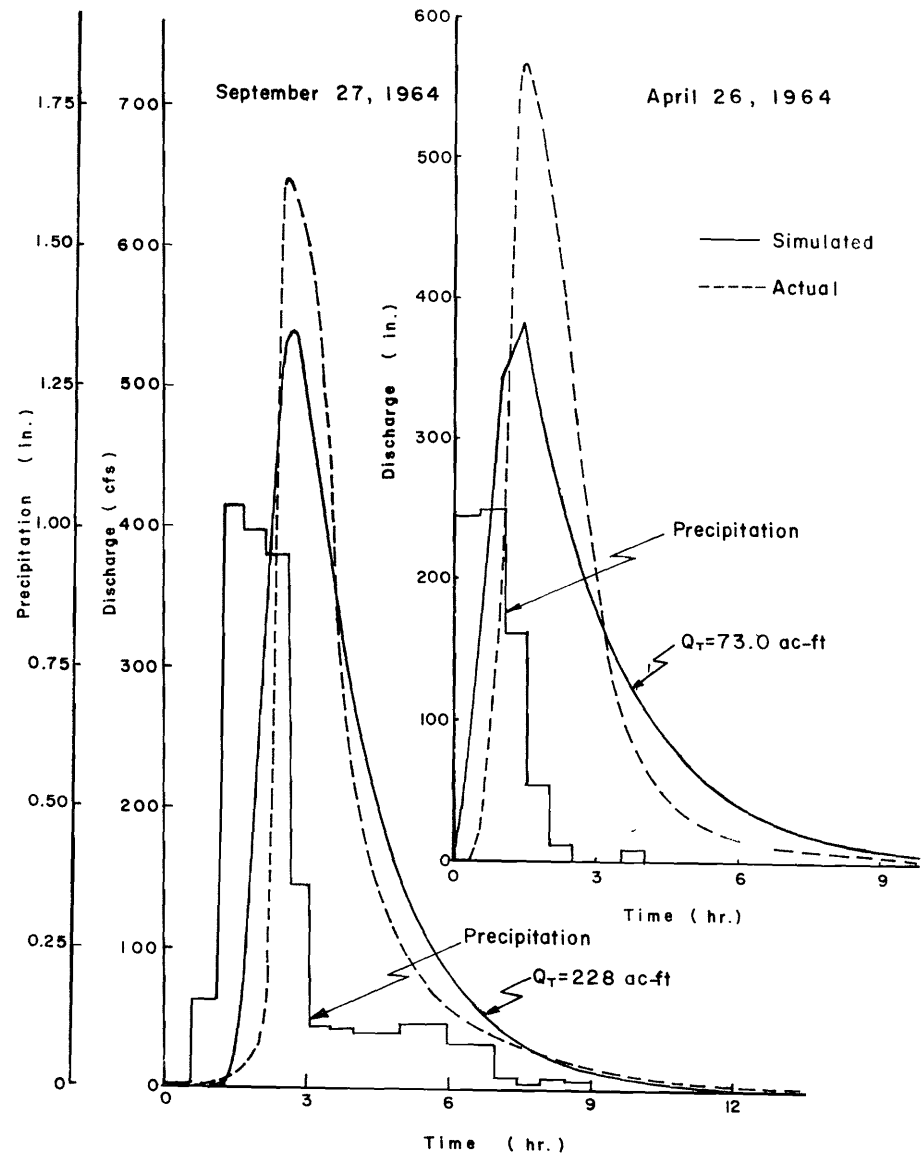


Figure 5.13. Comparison between the simulated and observed hydrographs for Wilbarger Creek watershed.

## Chapter VI

### SUMMARY AND CONCLUSIONS

#### Summary

This study represents an extension of the work of Narayana (1969) with respect to analog computer modeling of small urban watersheds. The specific objectives are described as follows:

1. To develop a method for estimating the hydrograph of runoff from a small urban watershed from a knowledge of input characteristics and the physical features of the drainage area, with the urban block forming the basic unit of study.
2. To develop and outline an efficient method of determining the various components of urban parameters from aerial photographs.
3. To investigate the applicability of the relations between urban parameters and watershed coefficients as applied to a rural watershed.
4. To apply the urban watershed model developed by Narayana (1969) to subzones within a watershed and, by routing outflows together, produce a more flexible model.

The method of determining urban parameters essentially consists of breaking a catchment area into homogeneous units which can be analyzed in terms of average sizes of roof area, road width, and ratio of sidewalk to roof area. Surfaces which may be of irregular size and shape, such as parking lots and industrial buildings, must be carefully measured on an individual basis. Working directly on large aerial photos greatly facilitates the case of data compilation.

The regression equations relating the watershed coefficients (interception, maximum and minimum infiltration rates, depression storage, and rise time) to the urban parameters of percentage impervious cover and characteristic impervious length factor for Waller Creek urban watershed were applied to Wilbarger Creek watershed, a comparable rural basin. This use of the equations tested them for the limiting case where percentage impervious cover was assumed equal to zero. Due to the meager amount of data, only four storm events were simulated with the runoff model.

The synthesis of hydrographs at intermediate points within Waller Creek watershed was accomplished by applying the urban watershed model to subwatersheds within the basin and routing the resulting hydrographs through the main stream channel. The routing technique utilizes the linear storage concept which assumes that storage is

directly proportional to the instantaneous discharge. Sixteen storm events were simulated over a period of eight years in which significant changes in urbanization of the watershed took place. Final outflows were compared with the observed flows recorded at the 38th Street streamflow gaging station. The effects of spatially varied precipitation inputs were determined at all subzone outflow points in the basin.

#### Conclusions

Based on the results of the previous chapter, the following conclusions were deduced.

1. The subwatershed model adequately predicts the final outflow hydrographs at the 38th Street Waller Creek gaging station.
2. The success in prediction of the final outflow hydrograph is assumed to carry over to the validity of the intermediate hydrographs synthesized upstream.
3. The range of application of the original urban watershed model has been extended to small subwatersheds within a basin.
4. Consideration of the main stream channel of a small watershed as a linear reservoir system in flood routing is a good approximation of the real process.
5. The ability of the subwatershed model to incorporate variations in spatial parameters, such as precipitation, urbanization, and channel improvements, while producing hydrographs at selected intermediate points of a watershed provides a useful modification of the original model.
6. The regression equations used to quantify the watershed coefficients of Wilbarger Creek watershed lack the accuracy formally achieved on Waller Creek. A more conclusive study of the runoff process on Wilbarger Creek requires additional rainfall-runoff data.

#### Recommendations

1. Improve the present urban watershed model.  
(a) Modify the urban watershed model to account for rainfall excess draining from impervious areas directly into man-made drainage systems (Figure 6.1). (b) Improve estimation of the effects on the



abstractive processes of time delays between successive storms. Better techniques of estimating antecedent conditions of isolated storms would also be included here. (c) Incorporate parameters into the model which reflect the influence of the man-made drainage system on the concentration of rainfall excess.

2. Digitize the improved subwatershed model. (a) Utilize 5-minute time intervals in synthesizing hydrographs in order to make full use of available precipitation data and to create outflow hydrographs nearly as continuous as produced on the analog computer. (b) Develop a program to optimize the values of watershed coefficients for all rainfall-runoff data.

3. Attempt to apply the digital model on an urban block scale. If the model proved adequate, then designers would have an alternative to the

rational method. More sophisticated flood routing techniques would necessarily have to be developed.

4. More precipitation and streamflow data should be collected as it becomes available. Additional aerial photographs as well as detailed layout plans of the sewer lines should be assembled where possible.

5. Other watersheds, preferably with longer more extensive records, should be modeled.

6. Application of the model to sediment transport and water quality problems should be undertaken whenever data becomes available.

7. Attempt to more accurately predict runoff from ungaged watersheds through the application of statistical methods.

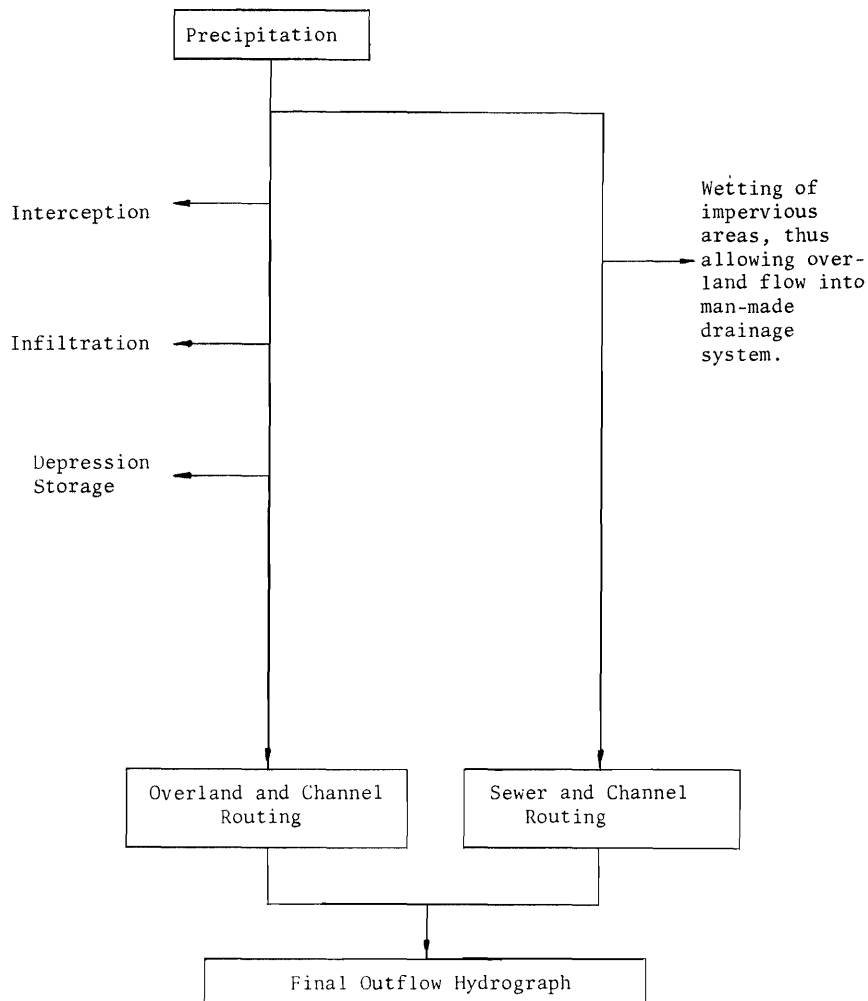


Figure 6.1. Proposed modification of urban watershed model to account for rainfall excess draining directly from impervious areas into a sewer system.

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# Appendix A

## Digital Computer Program for Computing Precipitation in Equal Time Intervals

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1* C PROGRAM FOR COMPUTING PRECIPITATION IN EQUAL TIME INTERVALS
2* C DT1DT2 DATE MONTH AND YEAR NTO TIME OF COMMENCEMENT OF STORM
3* C DELT(I) TIME INCREMENT K NO. OF OBSERVATIONS
4* C NO NO. OF TIME INTERVALS NGA GAGE NUMBER
5* C DIMENSION TP(500),T(500),P(500),DELT(10)
6* 1 READ(5,99)NO,(DELT(I),I=1,NO)
7* 99 FORMAT(I5,5F9.0)
8* READ(5,100)
9* 100 FORMAT(1H1,79H
10* 1
11* 2 READ(5,102)NGA,DT1,DT2,NT0,K
12* 102 FORMAT(I5,2A3,I9,I5)
13* WRITE(6,100)
14* WRITE(6,101)DT1,DT2,NT0
15* 101 FORMAT(1H,15X,5HDATE 2A3,8X,9HTIME ZEROI6)
16* WRITE(6,110)
17* 110 FORMAT(1H,7HGAGE NO3X,13HACC TIME(MIN)4X,19HINTERVAL PRECIP(IN)3X
18* 1,20HACC STORM PRECIP(IN))
19* READ(5,104)(TP(J),J=1,K)
20* 104 FORMAT(20X,8F7.2)
21* DO 324 J=1,K
22* T(J)=TP(J)*0.1
23* T(J)=I
24* T=I*10
25* 324 P(J)=TP(J)-P
26* 323 DO 15KK=1,NO
27* WRITE(6,360)DELT(KK)
28* 360 FORMAT(1H,20HTIME INCREMENT(MIN)=F4.0)
29* CR=1.0
30* AF=P(1)
31* AT=DELT(KK)
32* J=1
33* ASF=0
34* 3 IF(AT-T(J+1))5,7,8
35* 5 DT=DELT(KK)
36* AF1=AF
37* GO TO 10
38* 8 J=J+1
39* IF((J+1)-K)6,6,14
40* 6 IF(AT-T(J+1))9,7,8
41* 7 TAF=P(J+1)
42* GO TO 11
43* 9 DT=AT-T(J)
44* AF1=P(J)
45* GO TO 10
46* 14 TAF=P(K)
47* GO TO 11
48* 10 TAF=DT*(P(J+1)-P(J))/(T(J+1)-T(J))+AFI
49* 11 PI=TAF-AF
50* PIR=PI+0.005
51* ASF=ASF+PI
52* ASFR=ASF+0.005
53* TAF=AT
54* WRITE(6,106)NGA,NAT,PIR,ASPR
55* 106 FORMAT(1H,15,I12,2F22.2)
56* IF(AT-T(K))12,15,15
57* 12 CR=CR+1.0
58* AT=CR*DELT(KK)
59* AF=TAF
60* GO TO 3
61* 15 CONTINUE
62* GO TO 2
63* END

```

END OF CIVAC 1108 FORTRAN V COMPILATION.

D \*DIAGNOSTIC\* MESSAGE(S)

